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UNITED STATES DEPARTMENT OF AGRICULTURE

BULLETIN No. 831

Contribution from the Bureau of Public Roads
THOMAS H. MacDONALD, Chief

Washington, D. C. PROFESSIONAL PAPER August 25, 1920

SPILLWAYS FOR RESERVOIRS AND CANALS

By

A. T. MITCHELSON, Senior Irrigation Engineer

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NOTATION.

Unless otherwise noted, the various symbols used throughout this publication will have the following significance:

H—Effective head, in feet.

H_0—Loss of head at entrance of siphon, in feet.

H_1—Loss of head due to friction in siphon, in feet.

H_2—Loss of head due to sudden enlargements in siphon, in feet.

H_3—Loss of head due to contractions in siphon, in feet.

H_4—Loss of head due to bends in siphon, in feet.

H_v—Velocity head necessary to create velocity at the entrance, in feet.

V—Velocity, at throat of siphon unless otherwise noted, in feet per second.

A—Area of cross section treated, in square feet.

k—A coefficient in the formula for siphon discharge, ranging from 0.50 to 0.80 and usually assumed according to the materials of construction and the form of the siphon.

g—Acceleration of velocity due to gravity, =32.16 feet per second per second.

C—A coefficient, in the formula for weir discharge varying from 2.5 to 4.5 and determined from experiments on small model weir dams under low heads.

p—Height of weir in feet above the bed just upstream from weir and applied only in the Bazin formula.

h—Head, in feet corresponding to velocity of approach.

L—Length of weir crest, in feet.

Q—Quantity of discharge, in cubic feet per second.

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SPILLWAYS.

In its ordinary use the spillway is a device for removing surplus water from a reservoir or canal, in order that the water level within the reservoir or canal may not rise above the point considered safe or fixed upon as the maximum allowable height. It is distinguished from other types of wasteways by the fact that the surplus water passes over a crest or "spills" instead of passing through openings in the dam or canal bank.

The conditions necessitating spillways are many and they vary as to the requirement for capacity, the degree of safety factor demanded, by the extent or importance of the structure they protect, the location of the spillway with relation to that of the dam or canal embankment, and the functions they must perform in maintaining a more or less perfect control of the reservoir or canal in times of maximum inflow when a predetermined flowage line or freeboard must not be exceeded. This necessitates the provision for passing the highest floods over the spillway within the safe limit of maximum rise, and the conveyance of this water away without injury to the dam or canal embankments or to their appurtenant structures.

If a reservoir is to be located in a stream channel where the extent of inflow is not under human control, the spillway must provide for the passage of both normal and flood flow when the reservoir is full

and must protect the dam or embankment from being topped if it has not sufficient structural resistance to withstand the resultant shock, pressure, and vibration imposed upon it by overflow of considerable depth. In the case of earth dams or embankments it is necessary to provide a means for conveying the falling water away from the point where the embankment strikes the surface below it.

An elaborate method resorted to in accomplishing this purpose is shown in figures 1 and 2, Plate I, illustrating a system of drops and a stilling pool at the foot of the Lahontan Dam in Nevada. In any case it is necessary to provide a means of neutralizing the energy developed in the fall of the water over the dam at the point where it strikes the stream below so as not to endanger the structure by undermining it.

When the reservoir is not in the stream channel the conditions under which the spillway is to operate are greatly simplified in that the flow of the water is generally regulated either by diversion of all or part of the stream flow into a channel and thence to the place of application or storage (see fig. 1, Pl. II); or it may be supplied by long conduits always controlled by some system of headworks. Under such conditions, when it is necessary to provide spillways, they are designed to pass such excess of water as may reach the reservoir by the failure of the inlet works to function properly, the accumulation of surface water due to superdrainage, or a combination of the two.

Examples of spillways where there is regulated flow into the reservoir are numerous, and figures 1 and 2, Plate III, show a provision for an earthen dam in Mockingbird Canyon near Riverside, Calif., and the East Park Reservoir Dam of the Orland Project, near Orland, Calif., where there is a separate spillway and, in addition, a diversion dam and inflow control. The Roosevelt Dam shown in figure 2, Plate II, is an example of the type where the spillway is provided either in a center section or at the end, and provision made for the impact of the falling water where it strikes the stream below. The Holter Dam of the Montana Power Co., near Helena, Mont., a cross-section of which is shown in figure 4, is an example of an "ogee" type of dam provided with baffles and a water cushion to dissipate the energy developed in the overpour.

With relation to spillways in connection with canals, the same general characteristics prevail and their discussion may be taken up along the same lines though more properly following the cases where the inflow is regulated. In discussion of canals, spillways and escapes are generally included in the same class of structures, called wasteways, and although they are both used as protective agencies in the canal system, they differ somewhat in their functions. They properly should be divided into two classes in that an overflow spillway is

used to discharge the waters from a canal when it becomes filled beyond its normal capacity. When thus used the structure is essentially automatic in action and serves as a safety valve to prevent the canal from being overloaded and consequently overflowing its banks with disastrous results.

The escape can be either a spillway, a wasteway, or a sluice gate, the last named differing from the spillway of the overflow type in that it can be used for partially or completely emptying a canal. It is seldom automatic in action and requires some means of rapid operation, since it is usually required to operate immediately in case of a break in the canal banks below it. Only local conditions can suggest the advisability of locating one of these structures on a canal system or determine whether or not it is necessary, but the protection it affords the canal and its appurtenant structures usually warrants the cost. The number and distance apart at which wasteways should be located are dependent upon the importance of the canal, the topography of the ground above and below the canal, and the character of the service the structure itself is to perform.

A spillway acts automatically and operates to prevent a rise in the canal level beyond a safe freeboard. This rise may be the result of an excess of water coming from the headgates, surface run-off from lands higher than the canal, an excess flow produced by the closure of lateral gates above the structure, or by an obstruction falling into the canal, or the closing of an outlet or checkgate below the escape. A spillway must be designed to take care of the most adverse conditions resulting from these causes and in its design there must be considered the maximum quantity of water the structure will be required to discharge and the maximum rise above normal water surface which the canal will stand. These factors are often assumed and are rarely absolutely reliable.

The escape or spillway may include a checkgate as a part of the structure, in which case it must not only be capable of undertaking the duties of an overflow spillway, but those of an escape as well where the full capacity of the canal must be discharged. Ordinarily, however, the principal function of an overflow spillway is to discharge the surplus water above the desired normal canal capacity, whereas an escape is intended to spill the entire flow of the canal if necessary. An escape, on the other hand, embodies both types, but is intended for the protection of the system below it and to divert the entire flow of the canal to some natural drainage channel in case of a break or other emergency. It also may be used as a scouring or sluice gate, ridding the canal of deposited silt. The functions of the three may be obtained by building the structure in combination with a check immediately attached or a short distance below the spillway, escape, or sluiceway, as the case may be. Provision must be made to

discharge any excess over the safe capacity of the canal in countries where heavy rainfall occurs periodically, or where melting snow may affect surface run-off from the higher lands draining into the canal during the operating season. It may be desirable to collect part or all of such drainage and carry it as additional storage. Escape structures should also be provided with means for taking care of any surplus water in the canal produced by regulation of the flow in one part of the canal system without having provided for changes in all other parts of it. Their location will be desirable above points of questionable strength, above stretches of canal located on steep side-hill where slides are apt to occur, just above the intake of any important structure where there is any danger of erosion around its intake, or above any structure whose direct loss would not be material compared to the resulting damage to valuable property either connected with or foreign to the canal system. Capacity should be computed from the possibilities of combined flows resulting from conditions apt to develop above the structure.

The overflow spillway, wasteway, and sluiceway are similar in the common characteristic of requiring the addition of a wasteway channel to divert the waste water to a point away from the vicinity of the canal.

There are two general types of spillways, overflow spillways, and siphon spillways. The distinguishing features of the two types are that the capacity of the overflow spillway depends upon the length of the crest and the height of the water above the crest, and is increased in no way by the distance through which the water falls below the crest; while the capacity of the siphon spillway depends upon the area of the cross-section at the smallest part of the siphon and the difference in elevation of the water surface at the intake and outlet ends. In other words, the siphon utilizes the fall from the water surface in the reservoir or canal to the discharge end of the siphon to increase capacity, while the overflow spillway makes no use of most of this fall. The two types of spillways are discussed in detail, as follows:

OVERFLOW SPILLWAYS.

Overflow spillways are of three general types, the "ogee," the "steps," and the simple inclined type. Some dams combine the first two of these types by utilizing the top part as an ogee and having the lower portion stepped to break the velocity of the falling water.

Flow over a spillway is produced by the velocity resulting from the head measured from its crest to the surface of the water in the pond above. It depends entirely upon the stored head to increase volume per unit length, and, regardless of the height of the crest above the pool into which the water is spilled, no part of the fall below the crest level is effective either directly or indirectly. In

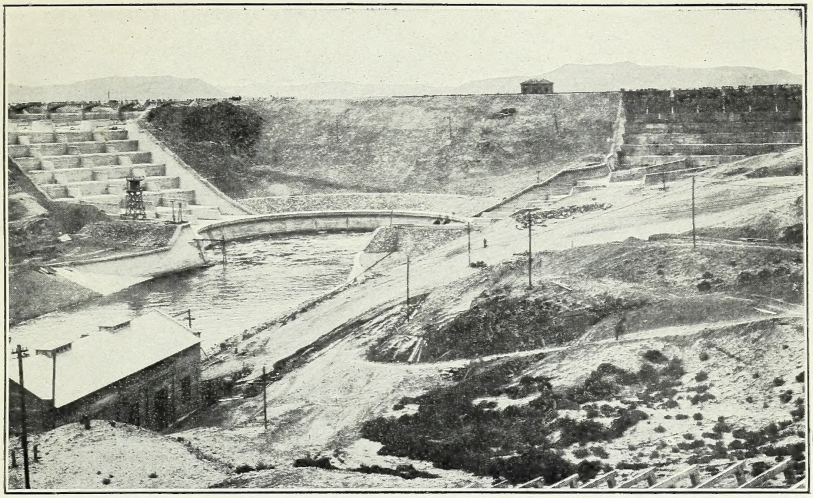


FIG. 1.—SPILLWAY OF LAHONTAN DAM, NEVADA.

Looking upstream and across stilling basin which receives the combined waters of the converging spillways.

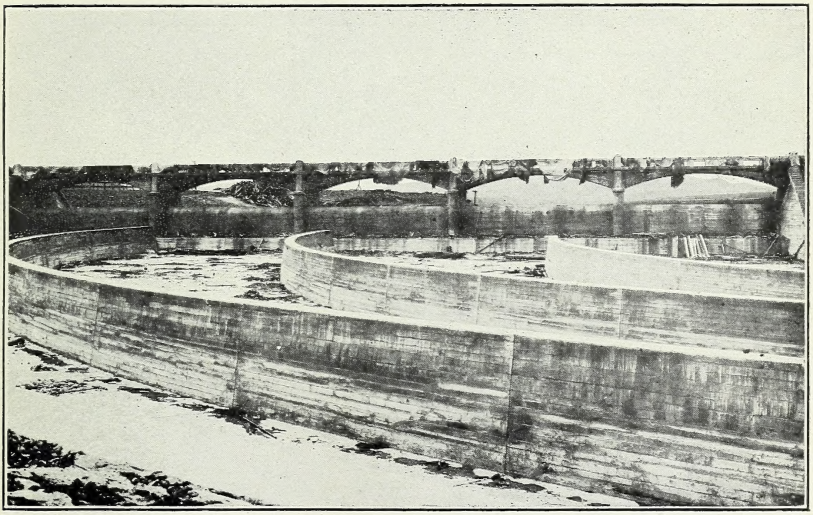


FIG. 2.—LAHONTAN RESERVOIR SPILLWAY, NEVADA.

Looking upstream and across the upper end of right-hand spillway. Shows training walls

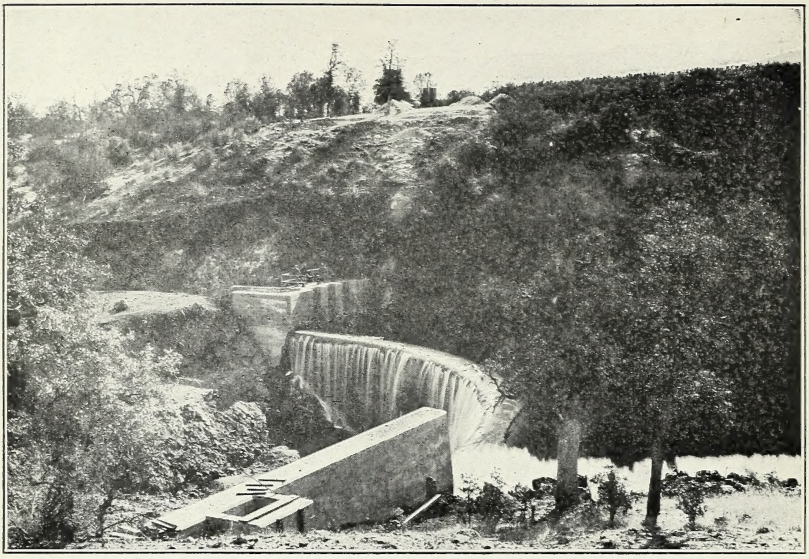


FIG. 1.—HEADWORKS, EAST PARK SUPPLY CANAL, ORLAND PROJECT, CALIFORNIA.

Shows diversion of part of waters of Big Stoney Creek into East Park Reservoir. Note gate control of supply.

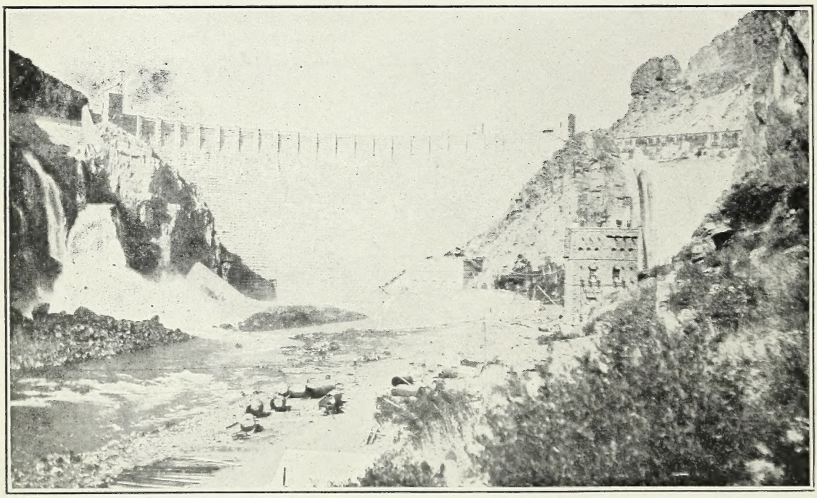


FIG. 2.—ROOSEVELT DAM, ARIZONA.

The two overflow spillways are shown at each end of the dam. Water for irrigation is discharged through the tunnel shown on the left and through the turbines of the power plant shown in right center.

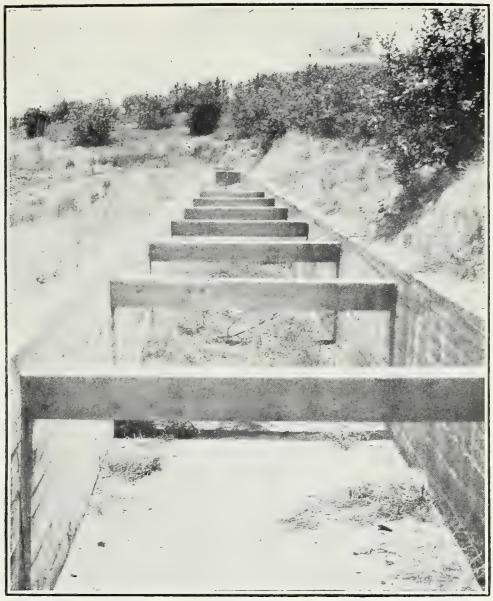


FIG. 1.—SPILLWAY OF EARTH DAM, MOCKING-BIRD RESERVOIR, GAGE CANAL CO., NEAR RIVERSIDE, CALIF.

Spillway crest is left edge of channel shown in cut.

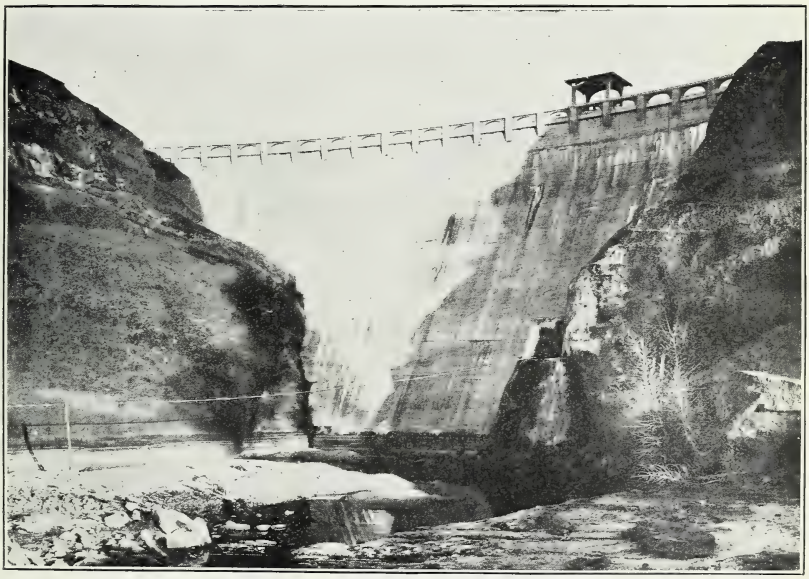


FIG. 2.—EAST PARK DAM, ORLAND PROJECT, CALIFORNIA.

FIG. 1.—DIVERSION DAM, WASHINGTON POWER CO., NEAR SPOKANE, WASH.
Built at the Little Falls plant in the form of an "L" to gain crest length.

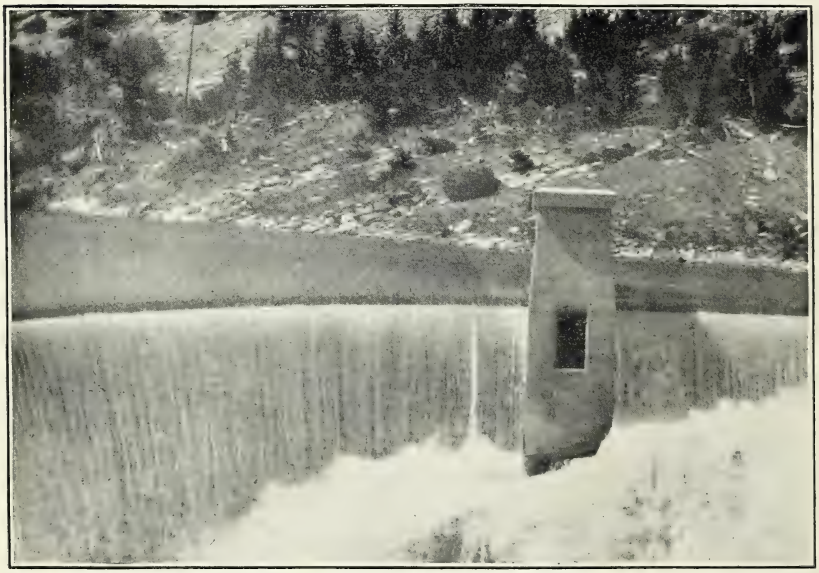


FIG. 2.—ARCHED TYPE SPILLWAY DAM, PISHKUN RESERVOIR, MONTANA.
Air intake tower shown in center of picture built to eliminate vibration due to vacuum.

other words, if a dam were 30 feet higher than the surface of the tail water at its base, and there were 5 feet of water pouring over its crest, of the 35 feet available head only 5 feet, or about 14 per cent, would be utilized to produce velocity. Since the two principal factors determining capacity are length of crest and head and the allowable increase in head is small, the only available means for increasing discharge is by increasing the length. One of these factors directly governs the other and only by knowing one of them definitely can the other be determined; or if the discharge of the stream is known and the head allowable as a depth of overflow is definitely fixed, the length of spillway to provide a specified discharge can be computed from the standard wier formulas by selecting the proper coefficient for the form of weir crest under consideration. These formulas and their corresponding coefficients are very indefinite because of the fact that they have never been proven to be accurate for heads of more than 4 or 5 feet over the crest of the weir.

Furthermore, in determining the required capacity of the spillway, a careful study must be made of the record or data relating to the precipitation or run-off of the catchment basin it is destined to serve and a liberal factor of safety added to the results of the computations to insure security in case of discharge greatly in excess of any known flood record, and in spite of the retentive capacity of the reservoir.

Long crests discharging thin sheets of water will give greater factors of safety than short spillways with greater depths, will afford less liability to fracture or do other damage to the dam as a result of the impact of the heavier volume of water, and will insure closer regulation of the pond level.

In the common formulas applied to determine spillway capacity, there are different elements to be considered in their application to the case under consideration, such as height of crest above the bed of the reservoir or canal upstream from the crest, length and width of crest, velocity of approach, correct determination of the head on the crest, and the number of end contractions, if any. All these influence the efficiency of the structure as a whole.

Possibly the formula most used in the United States is the Francis formula expressed in its simplest form as $Q = CLH^{\frac{3}{2}}$, and which is modified by the introduction of the elements referred to above wherever they apply. For instance, with velocity of approach this formula would be changed to read

$$Q = CL[(H+h)^{\frac{3}{2}} - h^{\frac{3}{2}}]$$

and if it considers velocity of approach and end contractions it must still be modified to read

$$Q = C(L - 0.2H)\,[(H+h)^{\frac{3}{2}} - h^{\frac{3}{2}}].$$

The Bazin formula is more complicated and may be expressed as,

$$Q=\left(0.405+\frac{0.0984}{H}\right)\left(1+0.55\frac{H^2}{(p+H)^2}\right)LH\sqrt{2gH}.$$

The constant C is dependent not only upon the width of the crest, but also upon its shape. The data resulting from the experiments to determine this factor are not consistent even for the same form of weir crest, but vary according to the head. They do, however, form a basis for the approximation of discharge over any form of weir under conditions ordinarily encountered.

As has been stated, the experimental data from which discharges are to be computed include heads of only from 4 to 6 feet, whereas flow over spillways varies from 0 to 14 feet deep. The effect of form of crest and friction decreases as the head increases, and it is also probably true that the coefficient for many ordinary forms of weir sections would tend toward a common constant value if the heads were indefinitely increased. This would be due to the more definite form of the "nappe" resulting. It is assumed also that flow over the spillway may be affected by the form of the nappe, which in turn varies when it discharges freely, merely touching the upstream crest edge, adheres to the downstream face of the crest, adheres to the top of the crest, adheres to both top and downstream face, remains detached but becomes wetted underneath, adheres to top, but remains detached from face and becomes wetted underneath; or it may be replaced by a depressed nappe having air imprisoned underneath at less than atmospheric pressure. A method of eliminating the effects of the last condition is shown in figure 2, Plate IV. Experiments have been conducted to determine the influence of the various conditions, and its extent, under heads of from 0 to 5 feet and over different forms of model dams, with crests ranging from a sharp edge to a width of 16 feet. Tables have been published giving the results of these experiments and coefficients for almost any form of weir crest to be applied for discharge computations of spillways under the various conditions.[1]

So much depends upon the judgment of the person making the assumptions, which in turn become fixed factors in the computations, that there very often appears the greatest difference in the resulting dimensions. The writer has had a case called to his attention where three engineers computing the dimensions of a dam as a suitable design for a certain location, and starting with the same assumed discharge, varied as much as 14 per cent in the discharge for a given crest length and head in a maximum discharge of approximately 100,000 second feet.

[1] The information concerning these tables and the discussion relative to weir discharge are based on the writer's personal opinion, resulting from a review of U. S. Geol. Survey Bul. 200, "Weir Experiments, Coefficients, and Formulas," by Robert E. Horton.

This is not surprising when it is seen that only one of the factors entering into the formulas cited is fixed by the form of the structure. It has been stated that in the process of gaging streams at dams the head is usually measured in comparatively still water in an open pond, but this is an assumption at best and must be corrected to conform to an assumed velocity of approach. In this connection it may be well to mention that the more complicated Bazin formula quoted in another part of this paper includes the correction for velocity of approach in the weir coefficient, and therefore the coefficient for a given weir is comparable only with the coefficient of another weir under the same head when the velocity of approach is the same in both cases. The Bazin formula also expresses the velocity of approach by means of the depth and breadth of the leading channel, which is rarely if ever of regular form so that the use of such an unreliable base is questionable. It is not the intention to discuss in this paper the relative values of the different formulas with their corresponding exponents and coefficients to apply in the various cases, but it is attempted to point out the great number of factors to be considered in the design of a spillway to keep the pond level within limiting bounds and at the same time perform its duty as a safety valve in times of excessive stream discharge. The uncertainty arising from the use of these formulas and the assumption of the maximum quantity of water to be discharged emphasize the need of using a liberal factor of safety in the protection of that part of the structure to be guarded. At best the result is merely an approximation.

Overflow spillways seldom are fully satisfactory on a canal system, since their crests are ordinarily built parallel to the center line of the canal, and to produce the effect of a normal canal supply the depth of water in the spillway over the lip would have to range from the maximum depth above normal surface in the canal at the upper end of the spillway to no depth at all at the lower end. Otherwise the canal would still be above normal supply beyond and below the spillway, and to produce full efficiency the crest of the spillway would have to be abnormally long. The writer has observed that the depth on the crest of most canal spillways is practically the same at both ends unless a long crest is built, which adds greatly to the cost. The use of an abnormally long spillway on a canal to discharge the water in excess of the normal supply, or above important structures where overflow would be particularly serious, requires the addition of costly and extensive training works and channels to divert and convey the wasted water to some natural drainage channel. Economy has been approached in some cases by the construction of the upper portion of these conveying channels parallel to the center line of the canal or spillway, or by building the spillways in a dentated form to gain crest length and at the same time reduce the area

of the wasteway channel at its upper end. One of these methods is shown in figure 1, Plate V, and another in figure 2, Plate V, and in cross-section in text-figure 1. The manipulation of the latter is described.

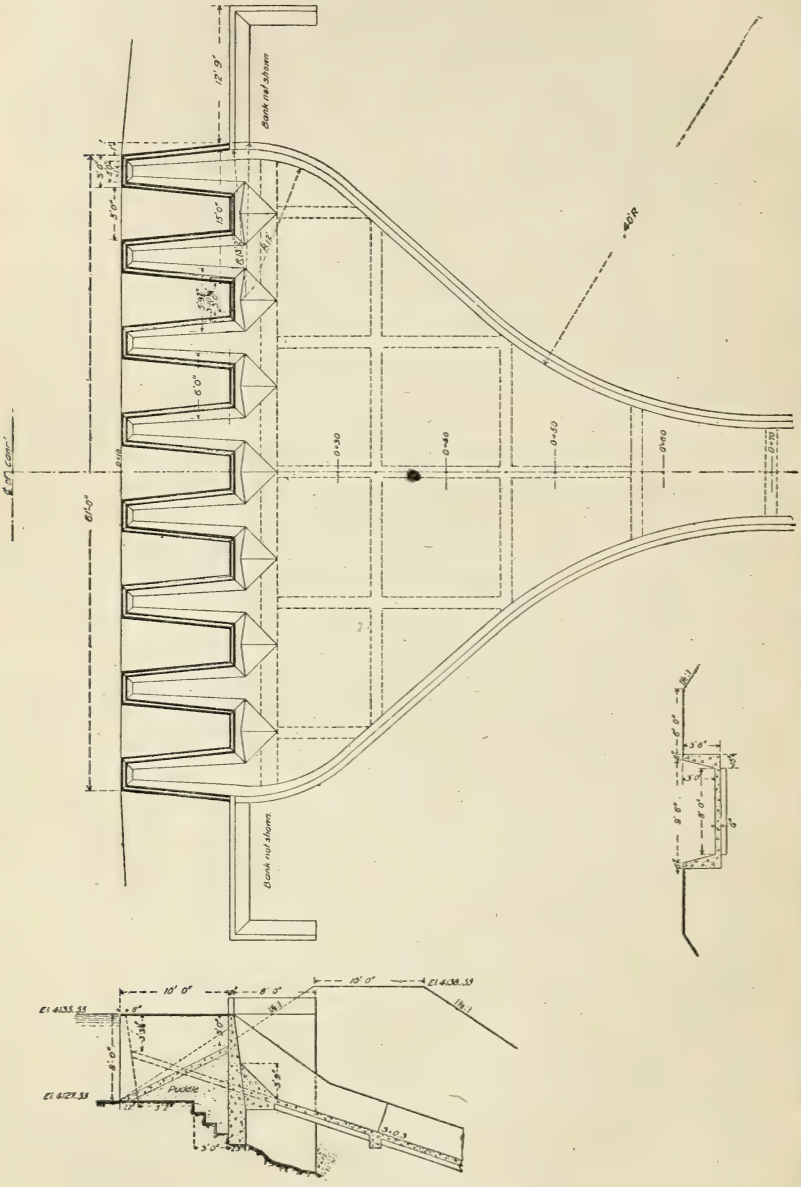


FIG. 1.—Spillway with concentrated crest length, Klamath project, Oregon. Built in dentated form to control flow line of Keno Canal where conditions would not permit of straight crest.

A spillway of reinforced concrete with a concentrated crest length designed to divert 600 second-feet of water from the Keno Canal of the Klamath project of the United States Reclamation Service has

been built at Klamath Falls, Oreg. (fig. 1). The water spilled is diverted to the Link River after it has been used to generate power. The fall from the canal to the river is 48 feet in a distance of about 170 feet. The canal would not permit of a rise greater than 1 foot above flow line, because it was built along a steep side hill and it was impractical to construct the banks of sufficient height to allow of any considerable rise. The conditions required a structure capable of spilling the waste water resulting from the proper regulation of a power plant above the structure without at the same time causing an excessive rise in the canal flow line. The sudden closing down of the entire plant presented an emergency in which the entire flow had to be taken care of quickly without exceeding the limiting rise and fall of flow line. An overflow spillway crest to satisfy such demands with its safe coefficient of discharge was calculated to be about 200 feet long, and to utilize as small a horizontal distance as possible was concentrated by indentations to a length of 61 feet as measured in a straight line along the canal bank.

The principal reason for concentrating the over-all length of the spillway was to provide a wasteway channel through which the water would be conducted over the earthen slope to the river after flowing over the crest. A section 3 by 8 feet with the slope of the ground at the point was capable of taking care of the discharge after it had been collected, but the concentration of crest length was to reduce, as far as possible, the dimensions of the upper portion of the collecting channel and therefore the cost.[1] The structure only resulted in this saving, as the same length of spillway had to be provided with consequent cost of construction. It is another example of the adaptability of the siphon spillway in cases where space and close regulation are paramount.

SPILLWAY CONTROL.

It may be desirable normally to carry the water near the maximum allowable level rather than sufficiently below that to provide for emergency flow. This is provided for by placing the permanent crest of the spillway sufficiently low to pass the maximum flow, and placing on top of this temporary or movable parts which will go out automatically or can be removed or adjusted in case of flood so as not to interfere with floating débris. The immediate removal of these barriers is of particular importance, because having provided an ample spillway it should be kept free of any obstruction and at all times ready for any sudden demands made upon it, regardless of other agencies which may be provided as additional facilities in effecting discharge. Inadequate satisfaction of these conditions has led to a great number of wrecked structures resulting in the loss of

[1] Description from an article in Engineering News, Sept. 9, 1909.

life and property. Sometimes legal requirements fix the flow line or it may be desirable to maintain a constant head in the reservoir. Long crests to keep the rise of the pond level within a minimum limit have been built and where the site did not permit of a straight-line crest, modification of the form has been made as in the case of the East Park reservoir of the Orland project, where crest length was gained by forming it in a series of nine semicircular arches resting against piers 8 feet wide. The arches have a radius of 13.5 feet and the whole structure is curved to a radius of 474 feet, the total length of spillway crest being 460 feet. It was designed to care for a discharge of 10,000 second-feet with a depth of 3.7 feet of water

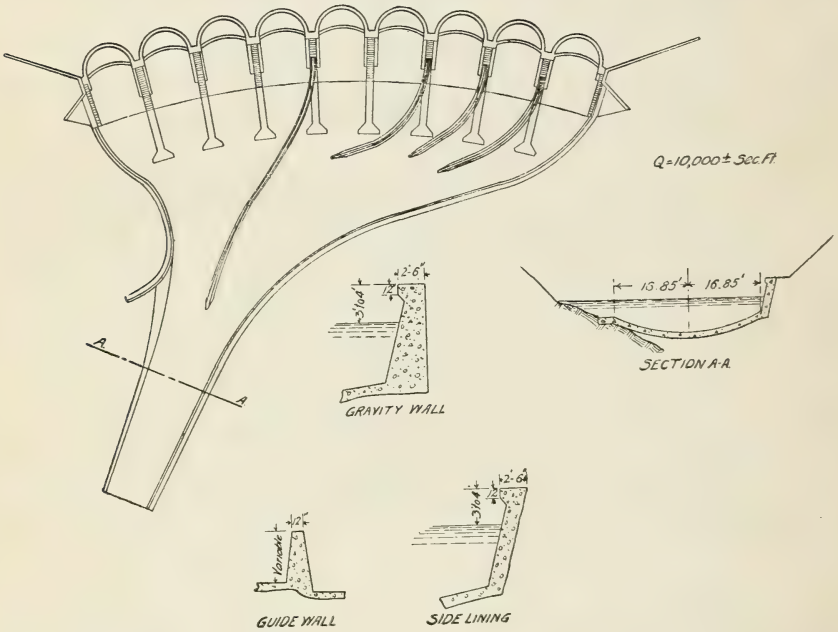


FIG. 2.—Plan and cross section of East Park reservoir spillway, Orland project, California. Form of dentated type of spillway composed of a series of nine piers, each 8 feet wide.

flowing over the crest. The spillway is located in a saddle about one mile from the dam and its outline is shown in figure 1, Plate VI, and in text-figure 2.

The Little Falls dam of the Washington Power Co., near Spokane, where the crest was built in the form of the letter "L" is shown in figure 1, Plate IV. The accomplishment of the above results, however, will add greatly to the cost of the structure if the longer crest has to be provided by the lengthening of the dam.

When acceleration is due to a shorter crest length and a resulting increase in the head to effect discharge, the devices serving to produce a regulated pond level must be under direct control. The

addition of such devices invariably serves the primary purpose of the spillway in facilitating the escape of the increased inflow, but lacks the automatic character which must be the distinctive feature of the true spillway. These agencies may be (1) flashboards in their various forms; (2) sliding gates or sluiceways; (3) Taintor or radial gates; (4) tilting or counterweighted gates; and (5) rolling dams or barriers. These are discussed in the order listed.

FLASHBOARDS.

Flashboards are the simplest but most unsatisfactory device. They permit of fairly close regulation of head due to the possibility of setting them to any desired width. The boards may be placed between stationary piers or may be held in place by tilting arms or braces set in line on the crest of the spillway so that they may not retard flow during flood, but will afford a certain pondage to provide surplus water from the time of minimum to that of maximum demand. This is shown in figure 1, Plate VII.

Another method is to have holes in the crest of the dam or spillway into which iron pins of a certain size and predetermined fiber stress per unit area are set and the boards fastened to these pins by staples. (Fig. 2, Plate VII.) The pins are calculated to resist pressure of the flashboards set against them up to a certain point, and when they bend the boards are released and go down the stream.

Such practice is not economical and is uncertain, due to the great variation in the strength of the iron pins. Where the storage of flood water is essential or desirable and the stored water is very valuable, this use of flashboards is not recommended, because they are generally leaky, and when they have gone out they can not be replaced until the freshet or flood has passed, making it impossible to store any of the flood water.

Flashboards are also installed as a permanent part of the spillway and are lowered automatically at different stages of water level, as are other devices hereafter discussed. The use of flashboards under any plan in which they must be removed one at a time is dangerous, because of the length of time it takes to remove them, since sudden floods might result in damage to the works before the boards can be removed.

SLIDING GATES.

Sliding gates are not adapted to use as emergency escapes for flood water, since they are not automatic in action, although ordinarily they are provided with mechanical means for quick operation. Ordinary slide gates are the slowest of all the types of gates to operate and are not suitable because of the destructive effects of vibration due to high velocity, and are extremely costly when placed in a structure on

a large scale to quickly discharge large volumes of water. The cost of installation and operation is a large factor in discouraging their use. They are useful as a means of entirely emptying a reservoir in case of accident or to scour out silt when they are placed low for these purposes, and they should be provided only as an additional safeguard and not when the passage of flood water in excess of the safe flow is required. Examples of outlet gates and the elaborate towers necessary for their installation are shown in figures 1, 2, and 3, Plate VIII.

TAINTOR OR RADIAL GATES.

Taintor or radial gates are usually set between piers as buttresses and on the crest of a spillway to provide additional storage. They can be operated with the expenditure of less power and time than the other types of gates. They have cylindrical surfaces and are so connected as to revolve on an axle usually connected to the curved face by means of arms or braces, the axle being set parallel to the center line of the crest on which the gate is mounted.

Two examples of radial gates are shown in figures 1 and 2, Plate IX. The curved or cylindrical surface may be of wood, steel, or reinforced concrete. Where radial gates are mounted to pass drift over the crest of the spillway in large openings and there is danger of the axle catching débris and clogging the passage, the long axle is eliminated and in its place bearing tubes are placed in the sides of the piers, and into these pins are set to form pivots or hinges. The radial arms of the gates are connected to these pins, so that the only forces to be overcome to lift them are the weight of the gates and the friction on the pins or pivots and on the faces of the buttresses or piers, because the pressure has been, of course, transmitted to a single point instead of being distributed over an extended bearing face, as in the case of the ordinary sliding gate.

The main objection to the use of the radial gate is that it lacks automatic control, unless its specifications of installation more properly conform to the class following, and the fact that it is seldom watertight where it comes in contact with the crest of the spillway or the sides of the opening.

TILTING OR COUNTERWEIGHTED GATES.

Tilting or counterweighted gates are among the number of ingenious types of automatic gates which have been designed and installed on spillway structures in this country and in Europe where it is necessary or desirable to maintain a relatively constant head under fluctuations, floods, or changeable use of water. Three types of these are shown in figures 1 and 2, Plate X, and in text-figure 3.

They are usually patented, vary in form of construction and in method of operation, and are satisfactory only to a certain extent.

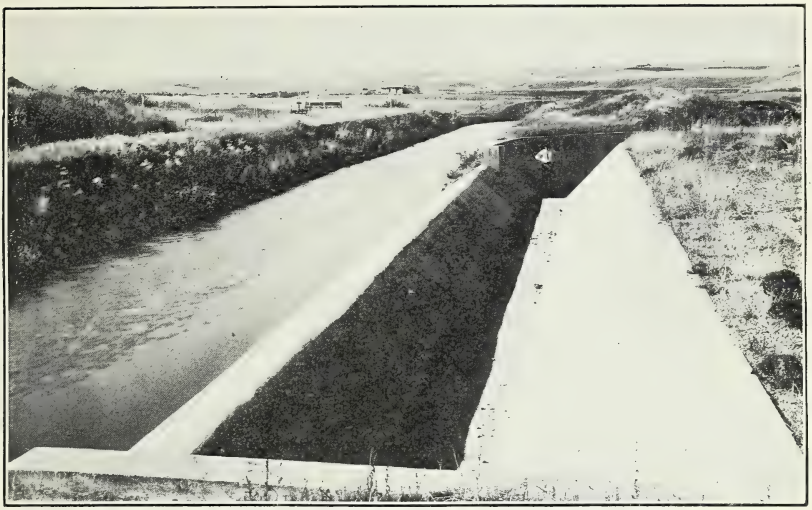


FIG. 1.—SAFETY WASTEWAY—LOWER YELLOWSTONE PROJECT, MONTANA.

Shows long overflow spillway on canal with upper end of wasteway channel parallel to center line of canal and crest of spillway.

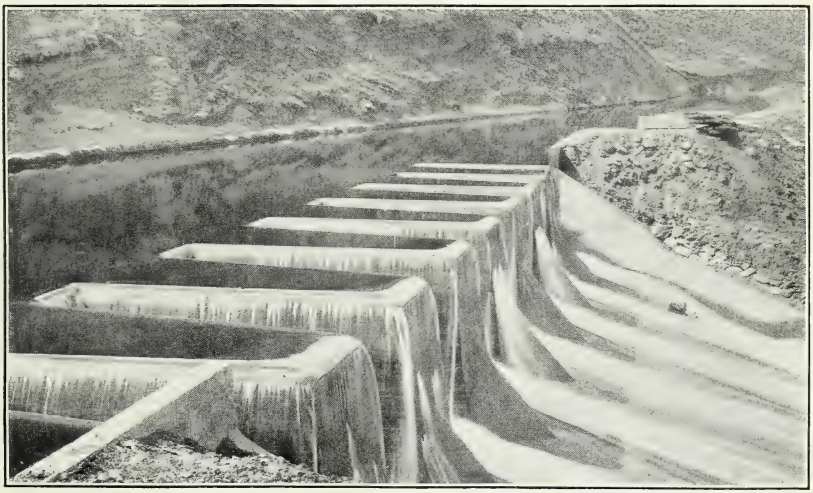


FIG. 2.—SPILLWAY WITH CONCENTRATED CREST LENGTH, KLAMATH PROJECT, OREGON.

This cut shows the details of the structure, and it also shows the high-water line on the upper bank of the canal.

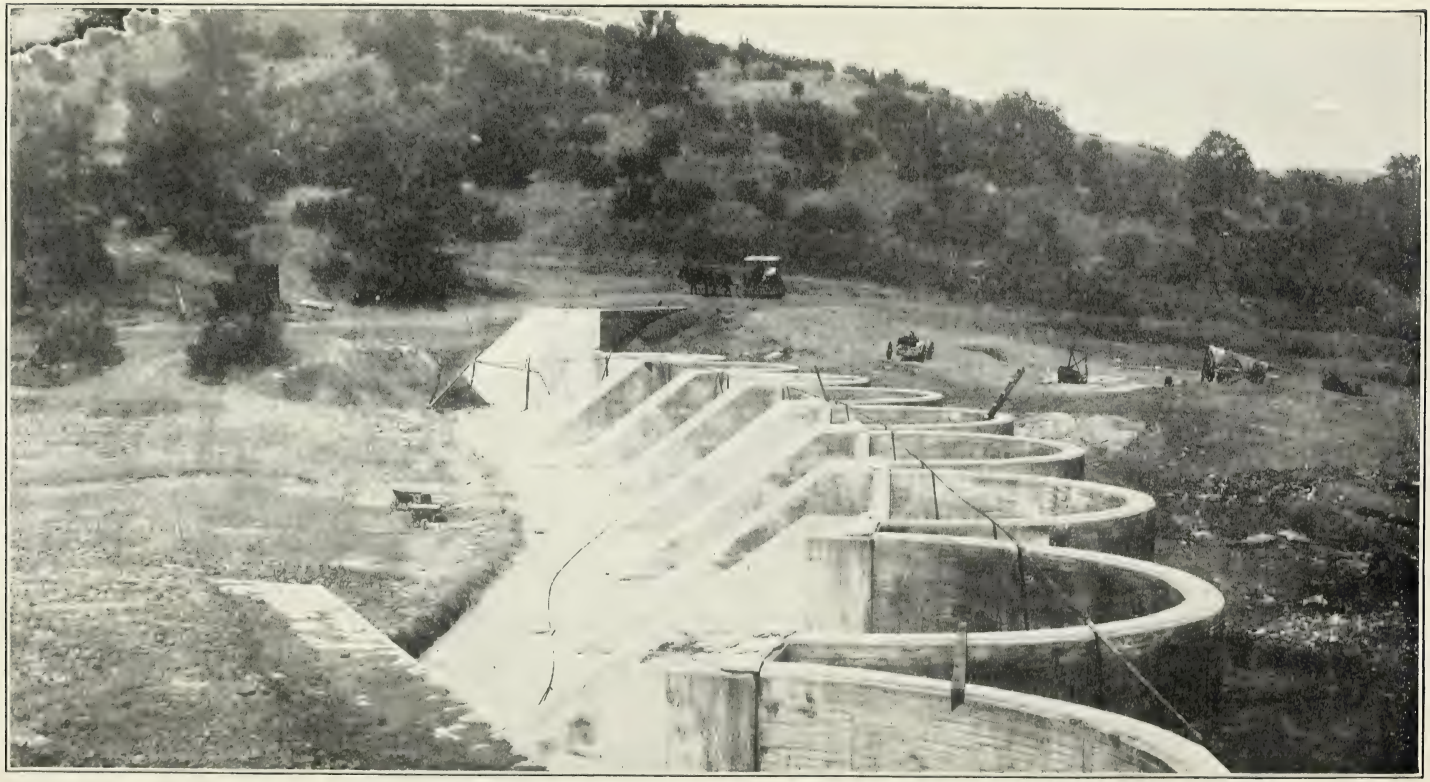


FIG. 1.—EAST PARK RESERVOIR SPILLWAY, ORLAND PROJECT, CALIFORNIA.

Shows view of spillway before curved training walls and wasteway channels were added to structure. This is one form of dentated weir intended to provide additional crest length.

They are installed as regulatory devices on numerous canal systems, where they work perfectly, because in such capacity they are kept in working condition by more or less frequent use of the operating parts. On spillway crests, where they have a limiting range of possibilities and their operation is less frequent, tending to make them less sensitive to the reaction of theoretically determined pressures, they are not as serviceable, unless the mechanical equipment necessary to operate them is more complicated and correspondingly costly. The moving parts must be maintained in the same perfect condition as that upon which their design was based if they are to function as they are intended, and such ideal conditions are rarely approached in actual spillway operation. Most types depend upon some system of counterweights to effect adjustment after they have been tilted by the action of the water pressure beyond a certain point. Or they may be so arranged that the water pressure on the gate or flashboard will cause them to react upon a cylindrical weight in such a manner as to make it ascend an incline by means of energy transmitted to the ends of the cylinder through ropes or cables; or others are arranged so that the tilting of the gate transmits power to the shaft from which counterweights are hung, such transmission being through chains or cables. Various methods are provided by which the introduction of cams or eccentric gearing may result in the compensation of the accelerated movement which might cause the slamming down of the gates or their too rapid rise.

An automatic form of spillway or floodgate has been installed on the Cedar River, near Nashua, Iowa. The design is one of the many patented types, but seems to be of simple operation. Figure 3 in section shows an outline of the structure, which is mounted on the top of a masonry dam 17 feet high and used for the generation of electric power. Each panel of the gate is 46 feet long and is so set that it will store 7 feet of water above the crest of the dam. To one end of a walking beam there is attached a reinforced concrete counterweight and a bar connects the other end with the floodgates. The gate is hinged at the bottom so that when the water rises above a predetermined elevation the gate is forced down and the counterweight is raised. As the height of water increases the pressure still further lowers the gate. When the gate opens the leverage between the counterweight and fulcrum increases, while it decreases between the fulcrum and gate hinges, and in this way overcomes the increased weight of water at every stage of gate opening.[1]

ROLLING DAMS OR BARRIERS.

Rolling dams or barriers have had their term of popularity and a number of them were installed on dams in this country. Possibly

[1] Description from Engineering News-Record, Aug. 9, 1917.

more of these have been installed on the more recently constructed spillway regulators than any of the other devices. Their popularity is based on the possibility of using them where long span is essential. The rolling dam is a patented type and consists of a large hollow cylinder, the diameter of which is usually made to correspond to the height to which it is desired to raise the pond level above the crest upon which the cylinder rests. Its length is made equal to the width

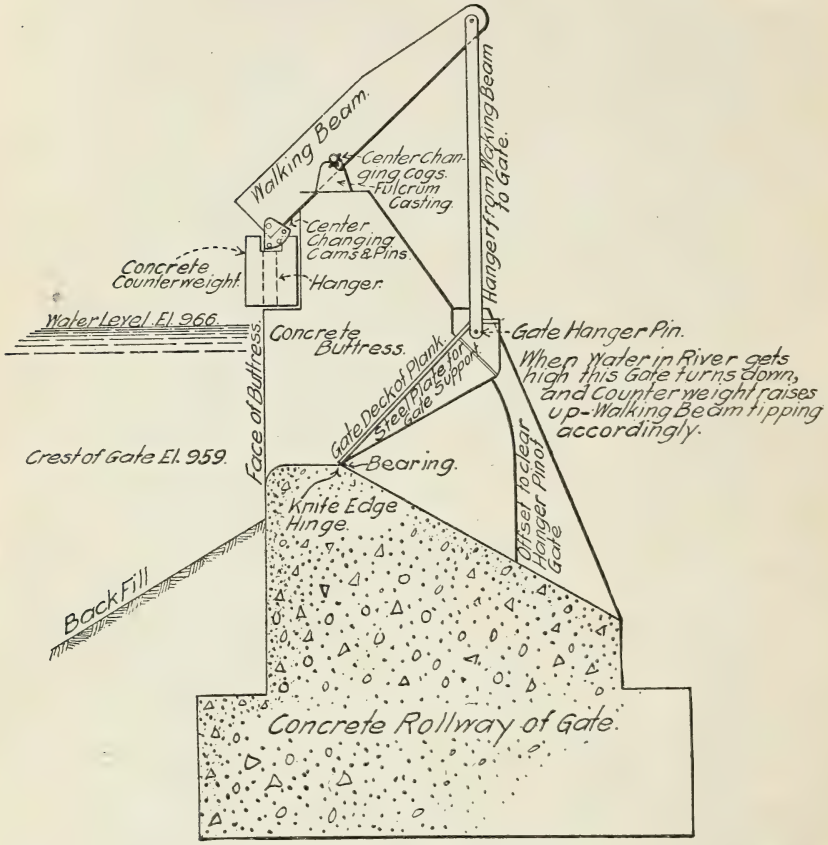


FIG. 3.—Concrete dam 17 feet high with flood gates, Nashua, Iowa. This structure on the Cedar River is equipped with automatic flood gates 7 feet high by 46 feet long.

of the opening into which it is fitted. As a general thing the material of which it is made is boiler plate riveted together and braced to withstand the strain to which the dam is to be subjected. The mechanical operation is provided for by having a gear engage a rack on an inclined abutment and by means of a sprocket chain wrapped around one end of the cylinder and connecting with the operating mechanism, the dam is rolled up or down the abutment as desired. The power is generally furnished by an electric motor or by winches. Various forms have been installed at Grand Junction, Colo., on one

of the spillways of the United States Reclamation Service at Boise, Idaho, and on numerous other spillway crests in this country and in Europe.

On the Grand River, near Grand Junction, Colo., the spillway of the United States Reclamation Service is provided with seven such rollers, six of which are 70 feet long and 10.5 feet high, and one 60 feet long and 15.33 feet high. They are set on the crest of a concrete spillway of the ogee type, 24 feet high and 537 feet long as measured between abutments, the spillway being designed to care for a discharge of 50,000 second-feet.

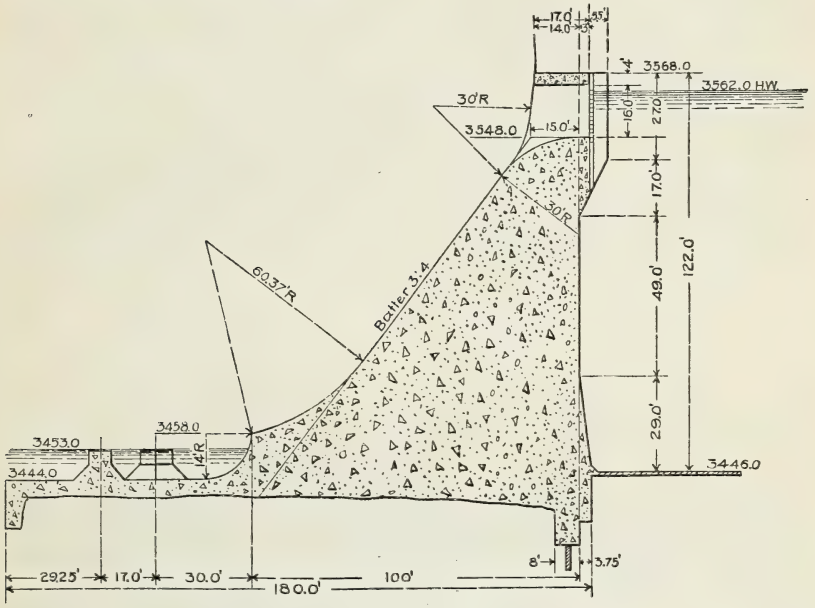


FIG. 4.—Holter dam of the Montana Power Co. Cross section of spillway of development near Helena, Mont. Shows baffles at toe of dam to neutralize the energy developed by the water pouring over the crest.

All these devices, as stated, are for spillway control and add nothing to its capacity or to its value as an automatic structure other than to lower the crest in times of flood and permit of raising the water level in normal times.

SIPHON SPILLWAYS.

As long ago as 1870 siphon spillways were used in Europe, but their operation required an ejector to cause them to function properly. Later designs making them as nearly automatic as construction conditions permitted were perfected and installed at about the same time by Heyn in Prussia, and Gregotti in Italy.[1]

[1] Engineering News, Apr. 20, 1911.

These were of comparatively small dimensions, however, and of a rather low degree of efficiency—estimated at from 40 to 50 per cent. A higher state of development was later obtained in France, where reinforced concrete was used and more efficient functioning effected by proper proportioning of the essential parts of the structure. Fear of the possibility of freezing weather interfering with the operation of the siphon spillways is said to be the principal reason why American engineers failed to adopt this type of structure, but a modification of the design led to the overcoming of this difficulty and to their rapid adoption at a number of points throughout the United States. The lowering of the intake leg well below the normal water surface, and the draining of the outlet sealing basin to prevent the possibility of clogging at that point, did away with the ice menace and also prevented the possibility of débris collecting or lodging in the throat.

There should be little argument against the use of the siphon spillway on canals or in connection with reservoirs where absolutely automatic control without the maintenance of mechanically operated devices is desired, and where there must be close regulation of the rise and fall of the water surface. Economy of cost, space, and rapidity of control greatly argue in favor of such structures, especially where laws require close regulation of the pond level, as in Italy where 8 inches is the limiting range. It is also most efficient where immediate response to sudden rises in water level is essential, because of the fact that it is brought into action to its full capacity with the rise of only a few inches, whereas the overfall spillway is not fully effective until the danger point is reached and then is dependent upon the stored head for acceleration of velocity.

The use in this country of siphons as spillways is much more recent than it is in Europe, some of the earliest examples being constructed in 1910 on the New York State barge canal.[1] Since then they have been used on the fore bay of a hydroelectric plant of the Tennessee Power Co. and on the power canal of the Mount Whitney Power & Electric Co. in California. They have been used still more recently on several projects of the United States Reclamation Service and on a number of irrigation and power projects in California, the notable cases being the Orland, Salt River, Yuma, and Sun River projects of the Reclamation Service; the South San Joaquin Irrigation District; the Sweetwater Dam near San Diego; and the Southern California Edison Co. at Huntington Lake, Calif.

As has been shown, the head utilized to produce flow in an overflow spillway is figured from the surface of the water above the spillway to its crest, although the actual head available, but which is not utilized, is much larger, being the total head from water surface

[1] Engineering Record, July 30, 1910, and Oct. 8, 1910. Eng. News, Nov. 17, 1910.

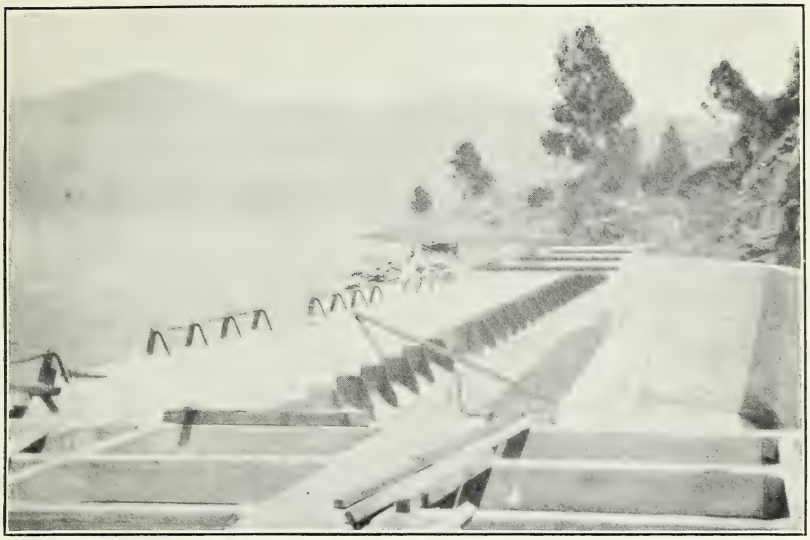


FIG. 1.—SOUTH OVERFLOW SPILLWAY, SWEETWATER DAM, CALIFORNIA.
Showing inclined braces along top of spillway crest to support flashboards.

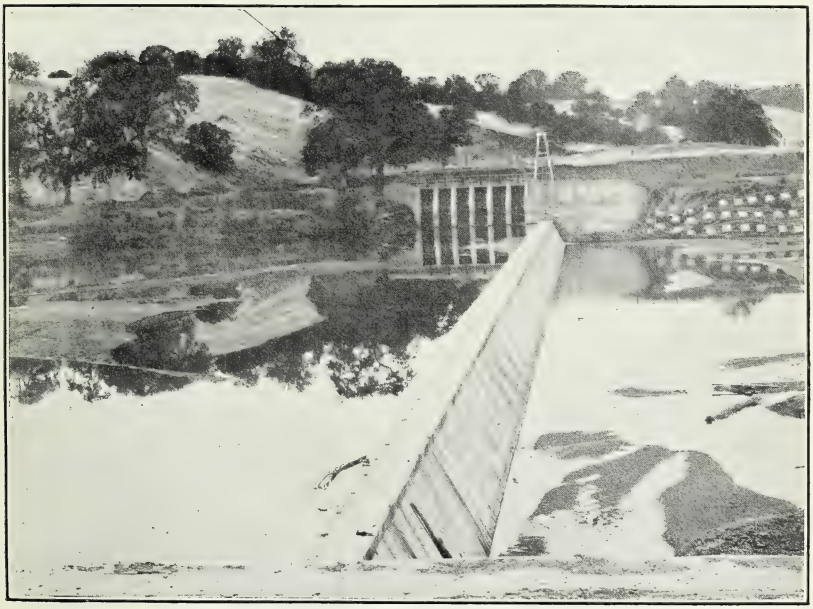


FIG. 2.—SPILLWAY OF DIVERSION DAM NEAR WOODLAND, CALIF
Showing holes along center line of crest to receive flashboard pins.

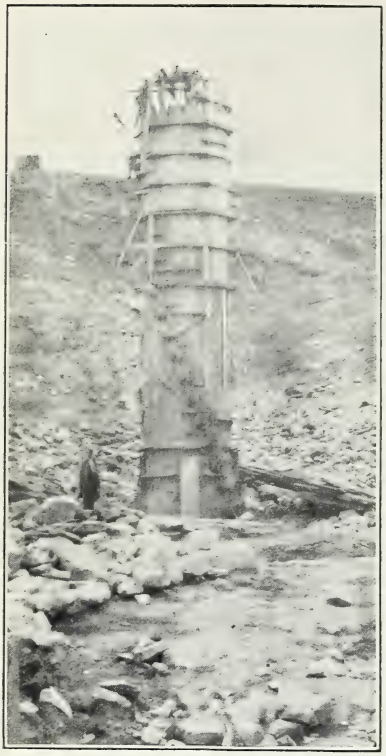


FIG. 1.—GATE TOWER, MAMMOTH RESERVOIR, UTAH.

This dam was washed out in the spring of 1917 because of inadequate spillway.

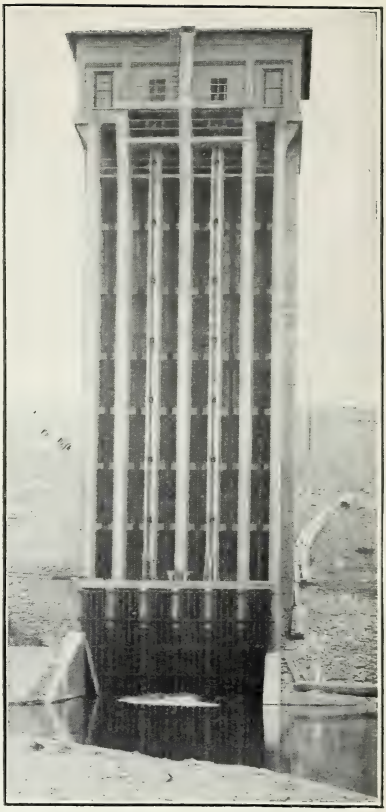


FIG. 2.—OUTLET GATES AND TOWER, LAHONTAN RESERVOIR, NEVADA.

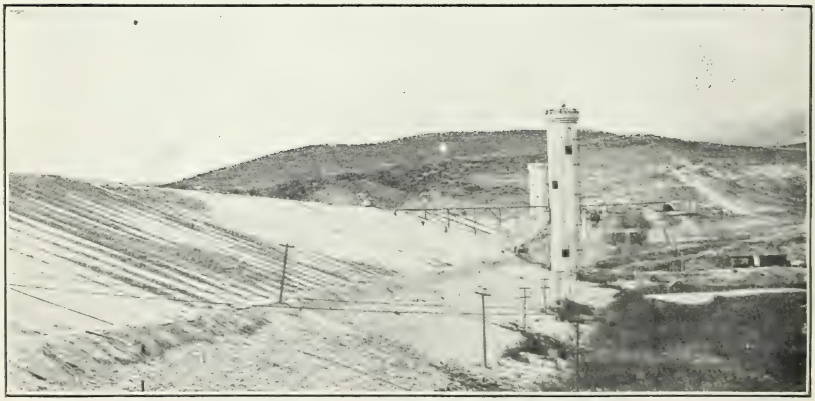


FIG. 3.—SAN FERNANDO RESERVOIR, LOS ANGELES SUPPLY.

Outlet towers shown at right center and at right end of dam.

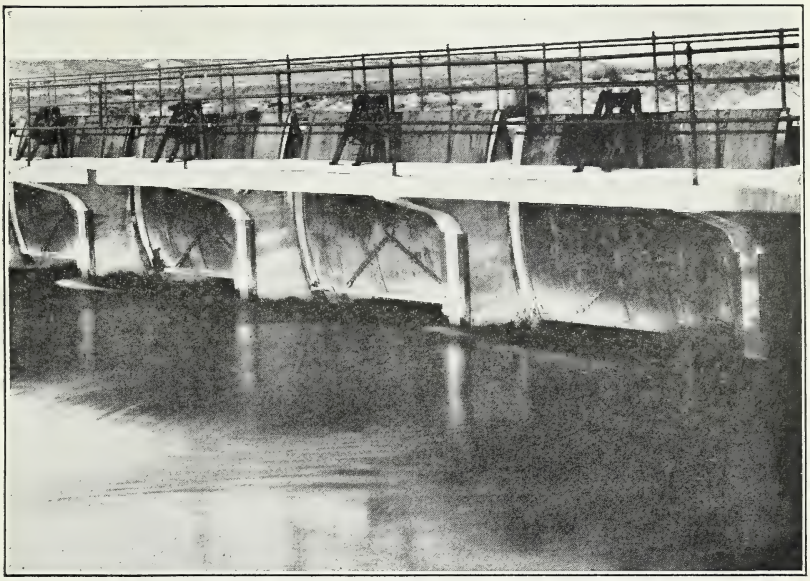


FIG. 1.—RADIAL GATES USED AS CONTROLLING DEVICE ON SPILLWAY CREST.

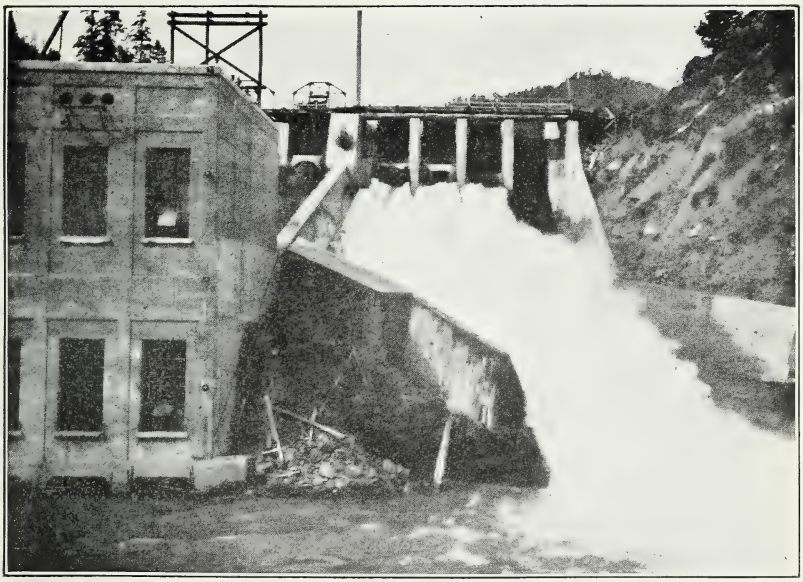


FIG. 2.—RADIAL GATES ON SPILLWAY OF POWER DAM.

Showing gates and wasteway of the Northwestern Power Co., Port Angeles, Wash.

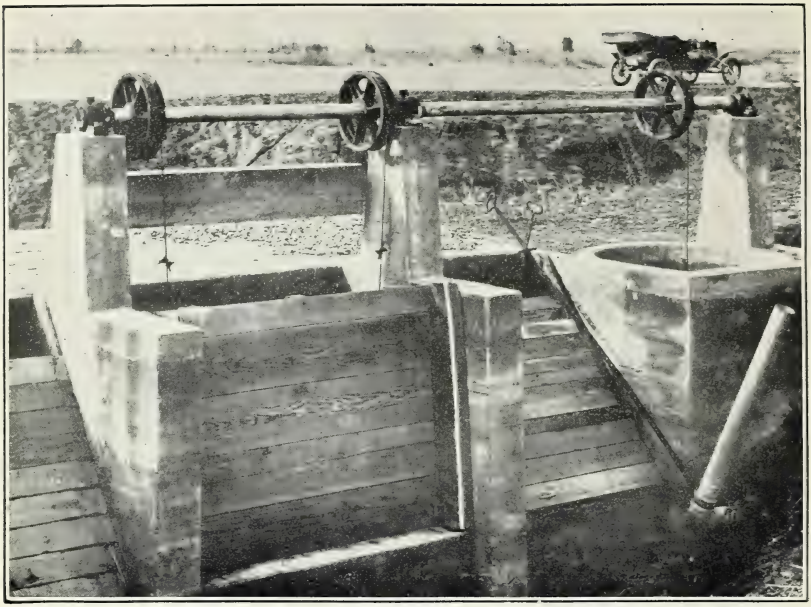


FIG. 1.—AUTOMATIC REGULATING GATE ON IRRIGATION CANAL.
Turlock Irrigation District, California.

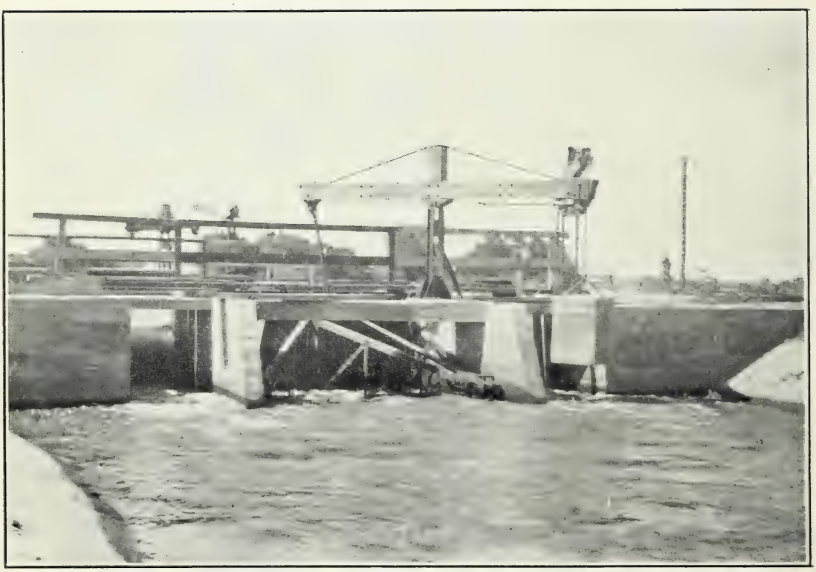


FIG. 2.—AUTOMATIC GATE ON DISTRIBUTING SYSTEM.
South San Joaquin Irrigation District, California.

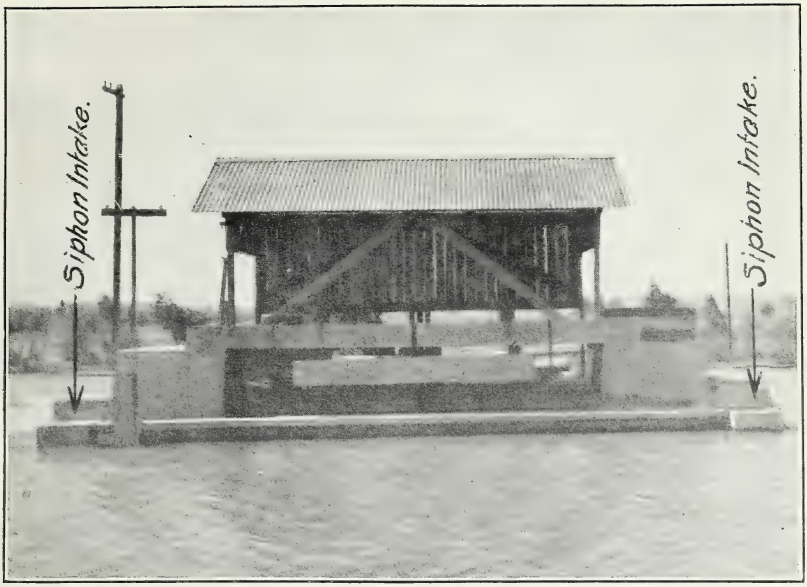


FIG. 1.—AUTOMATICALLY OPERATED GATE WITH SIPHON SPILLWAY ON EACH END.
Located on main distributing canal, South San Joaquin Irrigation District, near Ripon, Calif.

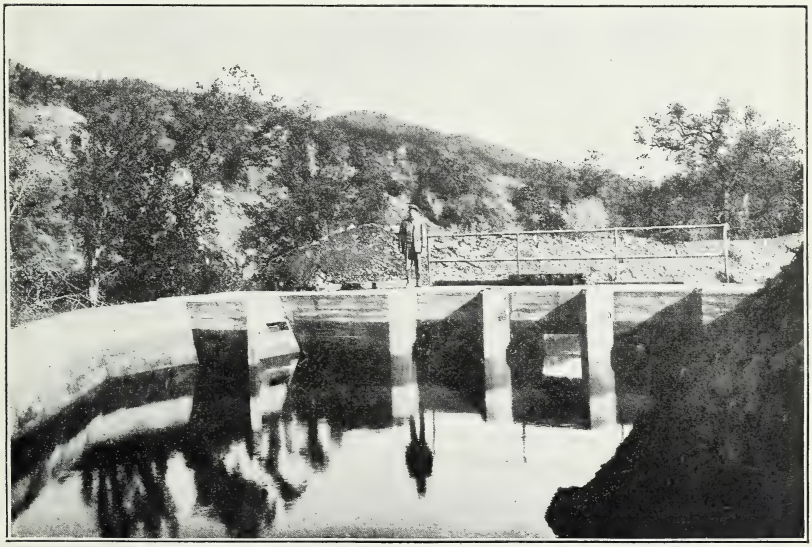


FIG. 2.—CHECK GATES AND INTAKE OF SIPHON SPILLWAY, ORLAND PROJECT, CALIFORNIA.

Intake lip of siphon is submerged and only air inlet shows at left of picture. Note close control of water level indicated by water line on structures.

above to water surface below the dam. Flow through a siphon is produced by the difference in the elevation of the water surface at the inlet of the siphon and the elevation of the water surface at the

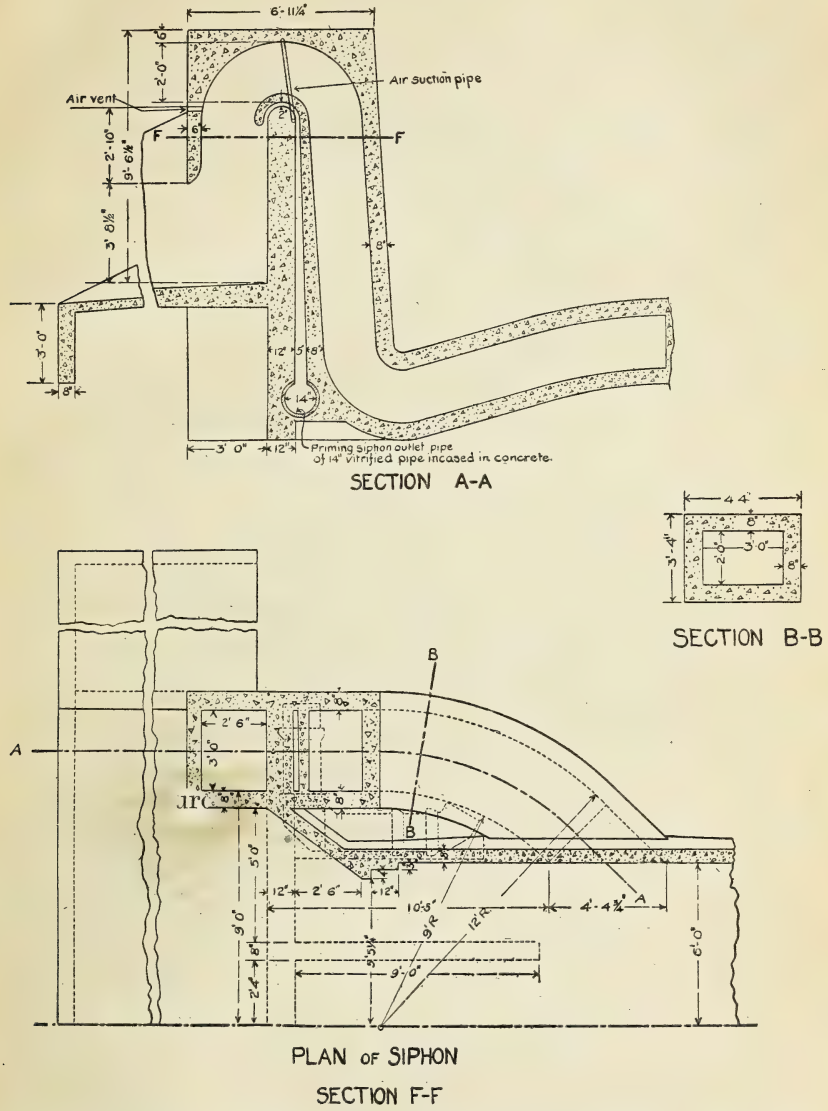


FIG. 5.—Cross-section and section plan of siphon spillway. South San Joaquin irrigation district, Ripon, Calif.

outlet end; or in case the latter is not submerged to seal the siphon, and the water has a free discharge, it is measured to the center of the outlet opening. In such cases the flow is treated as if it were through a short tube with various forms of inlet, and further limited

by a well-defined range of velocities at the throat which can not exceed that resulting from a complete vacuum. The tendency of design has been in most cases to assume that heads in excess of 34 feet were not adaptable to siphon installation, because it was accepted that the limit of vacuum draft had been reached at that point. As a matter of fact, siphons have been built with heads beyond 34 feet and in specific cases from 36 to 52 feet, but always, of course, with a resulting decrease in the coefficient of discharge due to added frictional resistance. It is generally accepted from a study of numerous designs that two forms of siphon tubes may be considered. The first of these will be treated as having throat and outlet of equal cross section, but not necessarily of uniform shape. Such a design has rarely been used in this country, but is rather common in Europe and is more or less followed in the development of the Ocoee River installation in Tennessee.[1]

The second type has a contracted throat section, expanding with a divergent tube as the outlet and further modified to overcome local losses of head, or contributing defects. There are numerous examples of the latter type, and their installation has been generally adopted in the United States and in Europe, so that most of the experiments conducted to determine efficiency, or the lack of it, have been on this type. Certainly all of the siphons which have been designed to date have been planned on the general principles of hydraulics, and still there is a dearth of information by which the designer may be guided in selecting the proper proportions for the vital parts, or in departing from present practice with any assurance of increased efficiency or the accomplishment of a desired purpose.

Nothing is definitely known of the relative head losses in the various parts of the siphon; the intake opening, the trance leg, the throat, the vertical or sloping outlet leg as the case may be, that due to shock or bends, or the curved discharge end. The effect on efficiency of a contracting or enlarging section beyond the throat is unknown, although such siphons have been built in this country. No experiments on working models have ever been conducted to determine other than the over-all efficiency, as valuable as such information would be when it is considered that the slightest change in the form of some part of a hydraulic structure will very often make the greatest difference in efficiency. Of course, it is in the long run only the efficiency of the structure as a whole which is desired, and the problem is solved, other than the value of having such information as will assist in arriving at the most economical design or in making designs conform to some additional specification as to speed in bringing the structure into operation and maintaining certain variations in allowable water surface.

[1] See description Engineering Record, May 16, 1914.

The formula $Q = kA\sqrt{2gH}$ is used as the basis for figuring discharge. While this expresses the quantity discharged, the accurate design of one of these structures will necessarily require more thorough search into the theory of the hydraulic features such as would enter into any tendency to retard flow and thereby reduce the efficiency of the whole structure. Coefficients, as has been stated above, have merely been assumed, and it is to be regretted that more data have not been developed to prove just how correct this practice has been. Forms and dimensions of the different functioning parts must necessarily be deduced from theoretical considerations, and on the results of data obtained from practical installations of such structures as have furnished any such information.

Theoretically, fixed rules have been adopted and practice has shown the advisability of having the overpour crest of the throat at the same elevation as the normal water level in the reservoir. For starting siphonic action the inlet must be sealed by the water as soon as the water in the reservoir rises above this normal height and climatic or other conditions may make this a matter of choice as to which of two methods will be used. The inlet lip can be made to extend down to the normal water surface or a very slight distance below it, so that when the water drops below this level the air is admitted and siphonic action ceases; or the inlet lip is extended to a considerable depth below the water surface, and the highest point in the crown of the siphon is connected with air inlets either fixed or regulated to admit air to the crown at the desired stage of water surface and break siphonic action. This latter case is used and is desirable when floating material may interfere with the proper action of the siphon. The contrast in the two designs is shown by text figures 6 and 7 as installed in Europe.

The discussions the writer has had with various engineers who have designed and installed siphon spillways have led to the belief that the assumed coefficients have been uniformly low; that refinements to a reasonable degree would lead to higher efficiencies; and that data to determine the reliability of coefficients of discharge and possibilities of future design must be obtained from existing installations. Any attempt to discuss the ideal proportions of the parts of a siphon spillway in order to approach the maximum efficiency would only revert to the general assumptions taken from formulas whose applicability has not been proven, but merely taken in all cases as being adaptable to this comparatively undeveloped structure.

In a series of experiments carried on at Throop College of Technology in California and at Fort Collins, Colo., several small models of siphon spillways were tested and some very interesting information obtained, proving that the conclusions from standard formulas to develop the proportionate dimensions of the various parts were

reasonably well founded. The Throop College experiments were conducted by Mr. R. N. Allen, a student of the institute, as a basis for a thesis, and were directed by Professors Ford and Thomas, also of the college, and Mr. Louis C. Hill, of Los Angeles. The Fort Collins tests were conducted by the writer, at the cooperative hydraulic laboratory of the United States Bureau of Public Roads and the Colorado Experiment Station and, of course, all of these tests must be given only such weight as would result from a laboratory rather than a working model. They will tend to encourage research

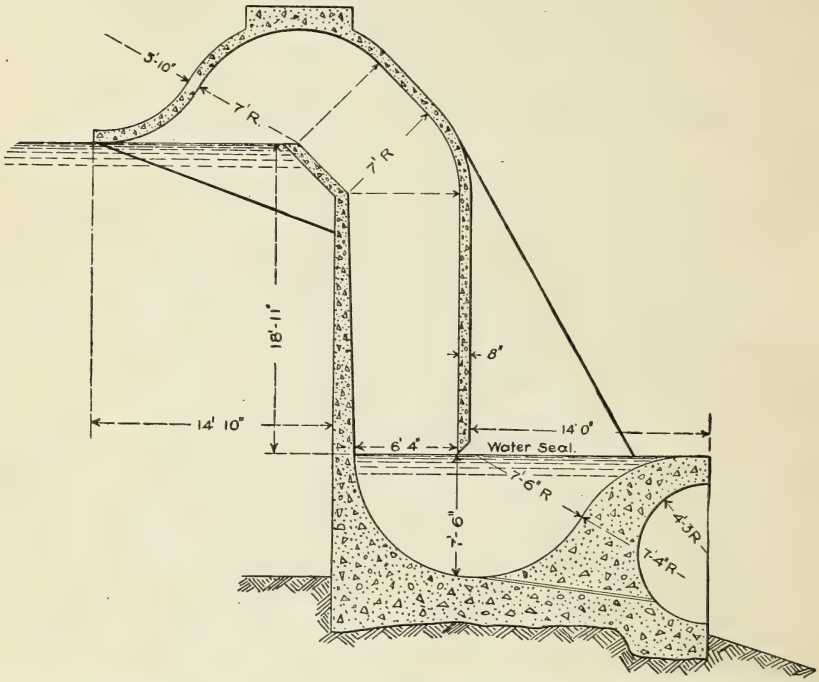


FIG. 6.—Cross-section outline of Lagolunga Reservoir siphon spillway. This is a typical example of the siphons installed in Europe where there is no trash or floating ice to clog the throat.

to develop the proper data from tests in a full-size structure, and suggest the study of such points as were not touched upon in the laboratory tests and will also stand as a guide to the finding of causes for the peculiar behavior in the smaller models. Great care was exercised in the design of the model siphons to obtain theoretically correct proportions for the intake, throat, outlet chamber, and outlet opening, both with and without the lower seal.

In the models for the Throop College experiments corrections were made for the friction coefficient of the materials as shown in standard tables. The outline of one of the models is shown in figure

1, Plate XII, and the arrangement of piezometer tubes and gage board in figure 2, Plate XII. A diagram of the testing laboratory and the setting of the siphons on the crest is shown in figure 1. Plate XIII.

The Fort Collins models were larger than those at Throop College and were designed as a miniature of the Phoenix siphon shown in figure 8 and hinged to permit of changing the shape of the outlet leg.

The tests were conducted to ascertain:

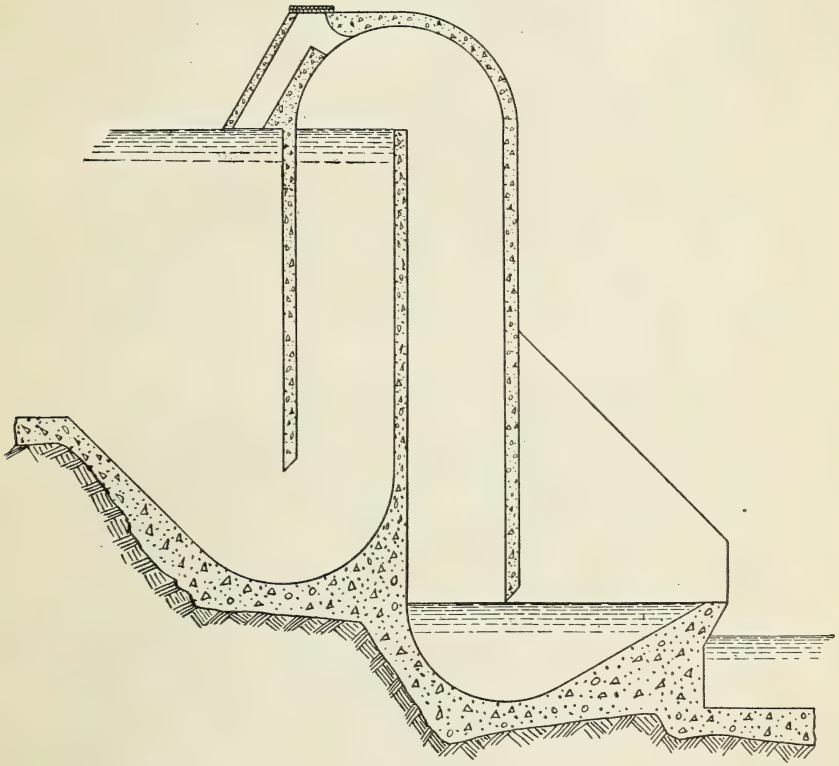


FIG. 7.—Outline section of type of siphon spillway built by Gregotti in Italy. Note submerged intake lip and surface air control intended for use where floating trash or ice interferes with operation.

(*a*) The theoretical and actual loss of head in the various parts.

(*b*) The effects of various shapes of parts upon head loss in that part and upon the general efficiency.

(*c*) The total efficiency of the structure and to determine a value for k in the formula $Q = kA\sqrt{2gH}$ for different shapes.

(*d*) The relation between the depth of submergence of the discharge lip and the depth of water over the throat to bring the siphons into action under different conditions of air inlet.

(*e*) The relation between the depth of submergence of the discharge lip and the speed of priming with various depths of water on the throat and various conditions of air inlet.

(*f*) Any additional relations which might become apparent.

COMPUTATIONS FOR DESIGN.

The calculations for the design of the models were varied to include a uniformly enlarging cross section of the outlet leg, then decreasing this to a uniform and finally to a converging section for the different tests. The formula $Q = kA\sqrt{2gH}$ was used, with A indicating the area of the throat in this case, but which may be taken as the cross-sectional area of the outlet leg, using, of course, the corresponding coefficient of discharge, depending upon the shape of the siphon and the extent of divergence or convergence.

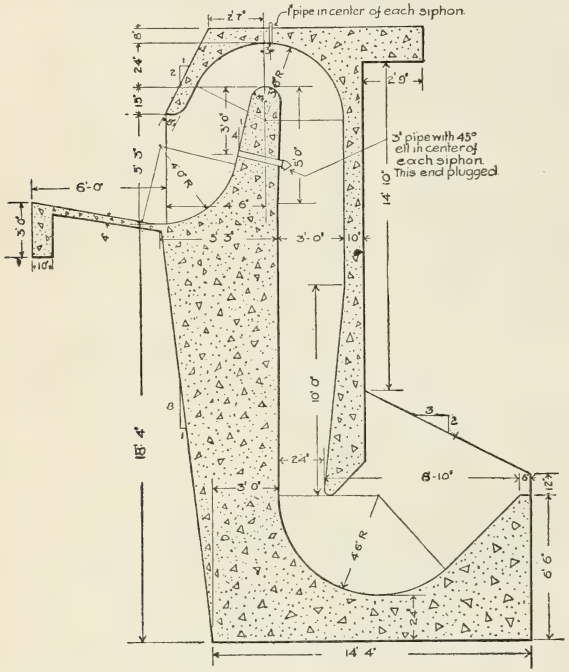


FIG. 8.—Cross section of siphon spillways, Arizona Canal, U. S. Reclamation Service, near Phoenix, Ariz. Installed to control the flow of canal at Arizona power plant. Note the converging outlet of the discharge chamber. Sketch shows air-control valves removed.

The effect of a properly expanding outlet leg is to increase the coefficient of discharge, calculated to produce results showing it to be greater than unity, and in siphon construction the extent of throat contraction is limited by a maximum throat velocity equal to that produced by a perfect vacuum.

Assuming that the operating head is relatively large, so that the outlet velocity will approach the highest throat velocity, the degree of expansion need be small, and in fact it may be advisable to make the outlet leg convergent, as was done on the final design of the installation of the Salt River siphons (fig. 8).

Data as to the effect of such practice have not been published. In the lower operating heads the outlet velocity may be low and it may be desirable to construct the throat area in a contracted form to such an extent that the throat velocity will approach the maximum

velocity as induced by a perfect vacuum. On one such installation known to the writer, where the most minute study was given to the computations for the design of the different parts of the siphon, and the maximum available head for producing velocity was 11.87 feet, the mercury gage used on a test of the structure indicated a partial vacuum equivalent to 15.60 feet. This was noted as suggesting that the siphon was acting in a manner similar to a compound diverging tube under pressure, and yielding a coefficient of discharge greater than 1 and which may even have approached $1\frac{1}{2}$ or 2.

As stated above, in a discussion of the computations for the proper proportioning of the parts, one may go thoroughly into the theoretical determination of their dimensions, but must come back to the realization that the data are too meager to justify any conclusions and surrender to the simpler formula based on the elements of cross section, velocity, and a predetermined constant.

If H represent the effective head—that is, the difference in elevation between the water surface at the inlet of the siphon and the surface in the tail water or the mid point of the outlet end (depending upon whether or not the outlet is submerged)—we may express the losses due to all causes in the passage of the water through the siphon as follows:

$$H=\frac{V^2}{2g}+H_0+H_1+H_2+H_3+H_4$$

In the above equation V is the velocity at which the water leaves the tube; H_0 is the loss of head at entrance; H_1 the loss due to friction; H_2 the loss due to enlargement of section; H_3 the loss due to contraction; and H_4 the loss of head due to bends.

Velocity of approach has the same influence on a siphon spillway as on a crest spillway, but this influence is so small compared to the influence of the head of elevation that it can be ignored.

Because of the fact that the outlet basin of most siphons is so constructed that the velocity head is completely dissipated in eddies, no mention is made of any recovery of velocity head. This formula, therefore, accounts for the elements which hydraulicians agree contribute to form a factor of efficiency for the structure as a whole.

No tests on other than laboratory models have been conducted to obtain correct results of the actual application of the factors, or to what extent they are influential.

It is assumed that an ideal inlet will be largely flared and then taper to the smallest cross section of the siphon, which is usually at the throat, because it is known that from tests on pipes of small cross section and of different materials the entry loss for a bell-mouthed intake will approach a value of $0.05H_v$. The value $0.25H_v$ has been assumed as the extreme limit for loss from shock or bend, but this has not been proven, in pipes of large diameter. Whether or not the assumption is correct can not be stated, and is here taken to apply where the radius of curvature is at least equal to the

height of the throat, and this coefficient is based on the velocity at the throat (V).

The friction loss in the outlet leg depends upon the material and class of workmanship therein, and is further dependent upon the cross section of the chamber with regard to the throat, which is assumed as constant and equal to the area of the throat.

RESULTS OF TESTS.

The writer has stated in another part of this paper that the only tests made to determine the losses on the different parts of the siphon are those of the small laboratory models, so that in summing up the results reference will be made to the tests of overall efficiency on working models built to discharge large volumes of water wherever such information is available. Some points have been brought out incidentally in these larger tests, indicating the value which may be placed on the deductions drawn from the laboratory work. Taking these points up in the order in which they are listed in a former part of this paper, the following is a summary:

(*a*) The theoretical and actual loss of head in the various parts of the structure as determined from the tests were not consistent for the various tests nor for the different models, but were of sufficient accuracy to warrant the use of the standard formulas until some more reliable data can be developed. The standard formula for the loss at entrance head $0.50H_v$ for the type of opening for which the formula was developed ran both high and low in the tests, and may be considered as holding good as an average, so far as any developments in the laboratory results are concerned. Friction loss in the structure was indicated as being negligible in the larger sections of the tube, and was heaviest at the throat or contracted section. It was so small as to be neglected in the results.

(*b*) The varied shapes of the discharge lip did not seem to affect the total efficiency, and since all of the models were of uniform design at the intake end, nothing developed in the tests at that point or in the bends from which to draw conclusions. No data from models of larger siphons are available with which to compare these.

(*c*) The total efficiency for the various models for different air-inlet conditions ran 0.84, 0.98, and 0.983 for the three sets of tests when grouped and averaged. Similar tests on larger models, but without the introduction of varying air-inlet conditions, ran from 0.644 to 0.805, and in a number of other siphons in this country and in Europe coefficients of discharge ranging from 0.70 to 0.82 have been found. These points will appear in the descriptions of the individual cases hereinafter taken up.

(*d*) The commonly accepted theory has been that the flow of water over the crest of the siphon would exhaust the air through the dis-

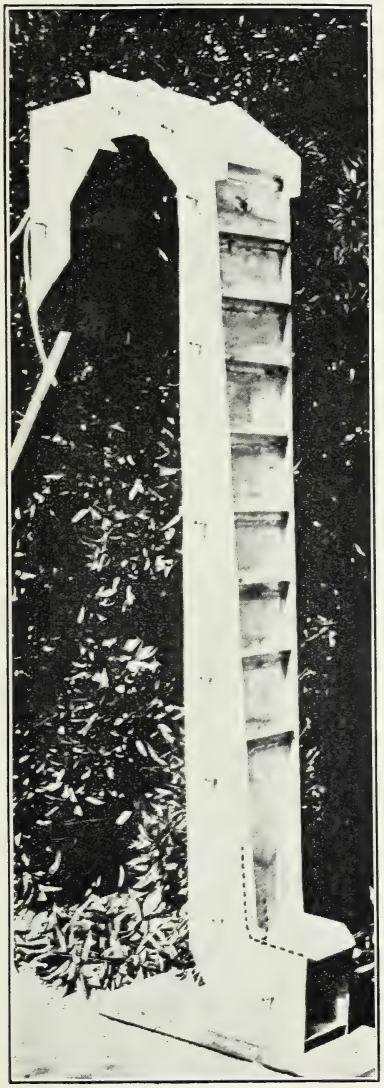


FIG. 1.—MODEL OF TESTING SIPHON.

Dotted line at foot of model shows approximate shape of discharge lip of the models used in the Throop College experiments.

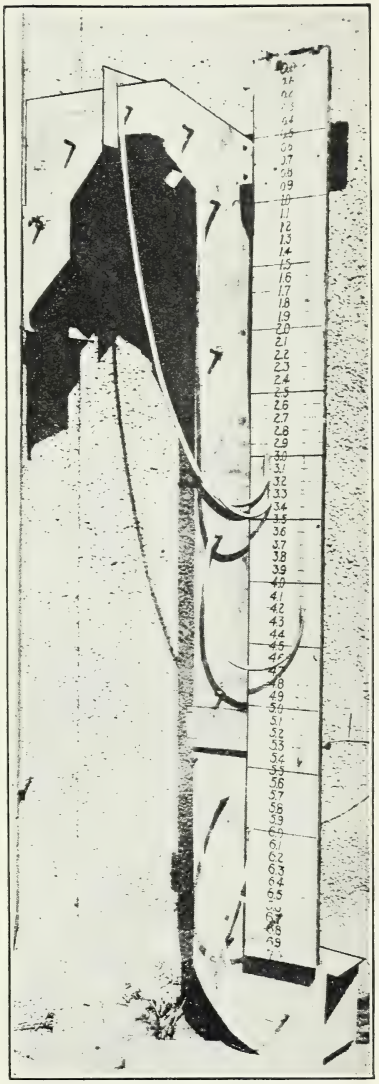
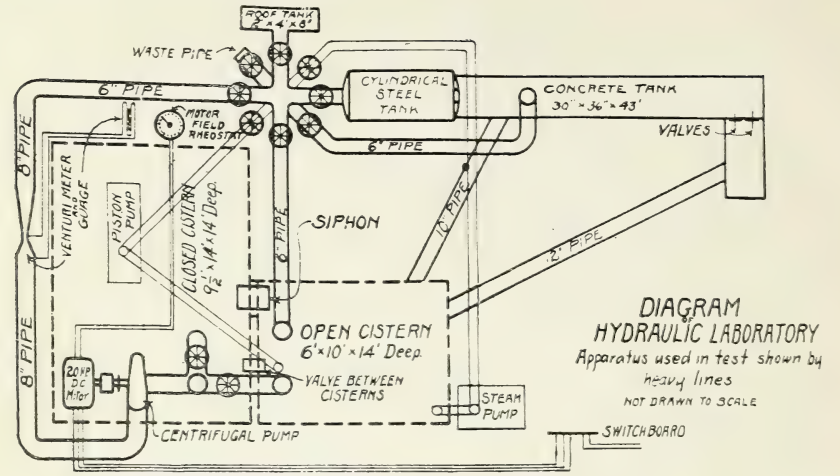
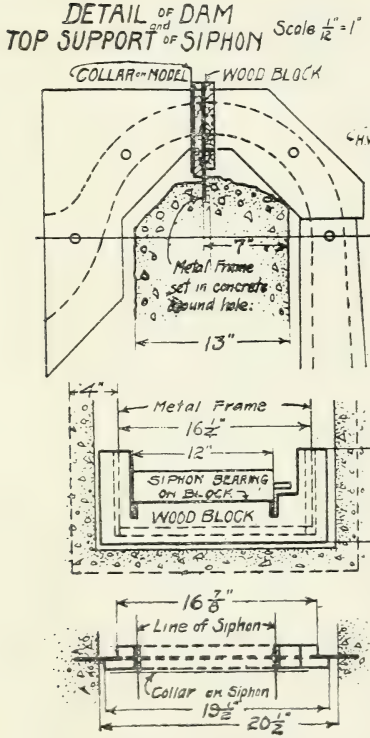


FIG. 2.—MODEL WITH GAGE BOARD AND SEVERAL PIEZOMETERS ATTACHED.

Connections for piezometer tubes were of ¾-inch brass tubing soldered flush to the inside face of the siphon.





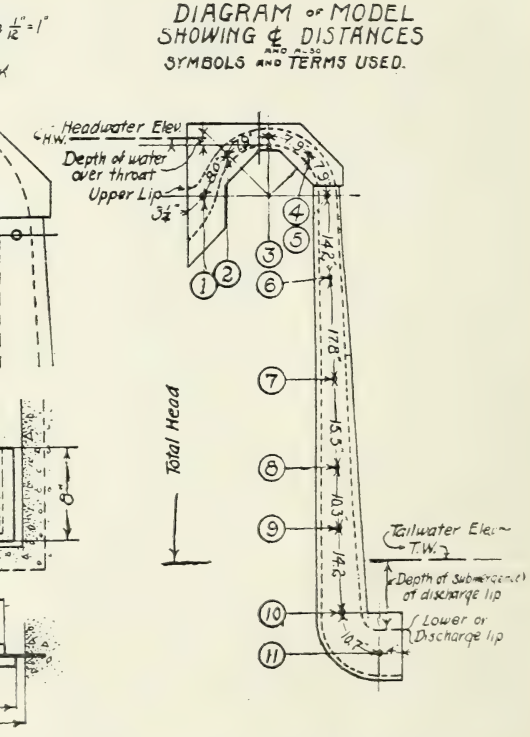


DIAGRAM OF TESTING LABORATORY AND SETTING OF SIPHONS ON CREST.

For experiments on models at Throop College (California).

charge leg and under the discharge lip, provided the outlet leg was sealed at that point and that the air becoming rarefied would encourage an increase in this flowing head until full siphonic action was accomplished. The tests on the laboratory model indicated a tendency to develop a back pressure in the crown by this rise of the water surface at the inlet, and the suppressed effort of the air to escape under the discharge lip when the seal was complete.

In fact, it was shown that with the discharge lip submerged 0.1 to 0.2 foot, and fully out of water, resulting in the tube being unsealed, the siphon primed easier and quicker than when the outlet lip was submerged to increasing depths to effect more complete seal. The tests on the Yuma siphons indicated that this is probably true, but the head on the discharge lip during the Yuma tests was not reduced sufficiently to disclose its action with sealing heads of 1 foot or under, or when flowing freely into air. The Ocoee River installation of the Tennessee Power Co., however, does bear out the tests of the small models, although it has been argued that the shape of the outlets of these siphons in itself forms a seal. Conclusive proof that a sealing basin is not necessary on siphon spillways or that the efficiency of the structure is not decreased by the absence of it, would lead to a great reduction in their cost and would permit of the elimination of another weak point in freezing weather.

(*e*) The placing of air inlets and the submergence of the outlet end are so closely related that the points are difficult of separation in summing up the results. One question to be solved is to determine the proper depth of submergence of the discharge lip to produce maximum siphonic action, and another is to determine the shortest period of time necessary to produce complete action under the same conditions. To enable the plotting of a curve to show these relations, various conditions were produced by the manipulation of improvised air inlets, made possible by utilizing the tubing for connecting the gage glasses. With all air inlets or outlets closed over the throat section, it is intended that the siphon will come into full action just as the water flows over the crest. The condition as developed with a complete seal of the siphon tube and with the water rising in the inlet and outlet legs produced an air compression in the tube which tended to retard the increase of head on the overflow crest, and consequently required a longer period of priming because of the resulting decreased flow of water necessary to produce expulsion of the confined air in the tube. The shortest period of priming was obtained by using the gage-tube connection at the throat as an automatic air valve, allowing all compression in the siphon to be relieved when the water rose to produce it, and then to be instantly closed when a reverse pressure was started.

(*f*) This suggested that the delays in priming were shortest when a relief valve was provided; that the heads on the crest of the throat

necessary to effect a complete priming were less under such conditions or when the outlet lip of the discharge leg was so deeply submerged as to set up reverse action in the expulsion of the entrained air.

Since the tests referred to were made a system of relief valves has been installed in connection with the battery of seven siphons placed in the Huntington Lake dam in California, the discharge from which amounts to 5,000 second-feet. The description of these siphons follows in another part of this paper.

SIPHON AT GIBSWIL, SWITZERLAND.

A siphon at Gibswil in Switzerland, built with a sloping instead of a vertical outlet leg, is described as consisting of a $\frac{1}{4}$-inch riveted steel tube tapering from 31.5 to 23.6 inches in diameter, the assumption being that the taper would tend to keep the water column from parting under the 52.48-foot head which was utilized. The inlet pipe was cut on a horizontal plane at the normal high-water surface and was incased in a reinforced concrete hood projecting 3.28 feet below normal water surface, so as to prevent the entrance of ice or floating débris. The air control, to break the action of the siphon and to prevent the water from being drawn down into the reservoir below normal surface, consisted of long narrow slots cut through three of the sides. When the water rises these slots are closed and siphonic action begins. A series of tests to determine over-all efficiency for this siphon gave a discharge of 98.9 cubic feet per second, but this was considered inaccurate and lower than the real efficiency of the siphon, because it was found that some of the air slots were not fully sealed. A maximum efficiency yielding 123.6 cubic feet per second was determined as a more accurate assumption of the real capacity. Computation of the end area at 3.03 square feet would give a corresponding velocity of 40.8 feet per second. The velocity due to 52.48 feet head is $V = \sqrt{2gH} = 58.06$ feet per second and thus the efficiency k is $\frac{40.80}{58.06} = 0.70$ as a coefficient of discharge. The computed friction loss in the pipe alone equaled 10.2 feet, which certainly is high and confirms the statement made in another part of this paper that the increase in the length of the tube beyond 34 feet would reduce the value k. In addition, this tube had the added friction produced by building the draft tube on a slope and thus making it longer.

In another siphon installation, also in Switzerland, the conditions to be overcome as the result of conflicting requirements of several plants and their water rights, were peculiar. There was a spinning and weaving mill operated by hydraulic machinery, the tail water from which, up to a maximum of 56.5 second-feet, was appropriated by a twine plant farther down the stream. The discharge from the first plant in excess of the 56.5 second-feet had to be led over a weir

spillway 230 feet long and through two sluiceways into the canal of still another plant of a hydroelectric company. Obligations fixing the disposal of the water over the spillway provided that the head must not exceed 2 inches, and since on the one hand the discharge was variable with 160 second-feet as a maximum, the demand of the power plant for water was also variable, so that close regulation and continuous attention were imperative. The power company had built a 20-foot spillway, but the water often rose 10 inches above the permissible stage and thereby interferred with the operation of the plant above. To eliminate the disadvantage to all concerned, the twine mill built a siphon for a computed discharge of 70.6 second-feet and the power company built one for a computed discharge of 88.3 second-feet. The latter was built on the old spillway, which was broken through and covered with a reinforced concrete hood. The head between pond and tail water was only 4.92 feet, but the completed structure gave a test discharge of 91.8 second-feet. The cross-section of the siphon was uniform and of an area of 9.47 square feet, giving an actual velocity of 9.7 feet per second and corresponding value of 0.55 as a velocity coefficient. This case is described in detail to illustrate the adaptability of the structure and to emphasize its value in just such cases of conflicting regulatory requirements, of which there are many.[1]

TENNESSEE POWER COMPANY'S SIPHONS ON OCOEE RIVER.

Where the canal of the Tennessee Power Company crossed a ravine it was necessary to design a structure capable of spilling from 1,300 to 1,400 second-feet. The structure must also take care of a rise of 1 foot in water elevation in a period of 8 seconds in case the power plant just below it should be suddenly shut down or in case of stoppage of flow due to slides or other obstruction just above the forebay. The 1 foot referred to was the limit of freeboard in the canal at the point. An overflow spillway capable of satisfying the conditions would have been 400 feet long, and since the available space in which the structure had to be placed was insufficient, a battery of 8 siphons, each having a throat cross-section of 8 square feet, (1 by 8 feet), was installed, with the addition of a sand gate. The operating head on four of the siphons was 27.2 feet and on the remaining four 19.2 feet and the draft resulting produced an increased velocity corresponding to the difference in elevation between the water surface in the forebay and the center line of the siphon outlets, minus the usual losses due to entry, bends, friction, etc. The vertical draft tubes change gradually from the throat cross-section at its upper end to 4 by 2 feet at the lower end, where it connects

[1] Data taken from a paper by Herr J. Huber and published in Engineering News, May 3, 1913.

with the flaring outlet. In each unit the inlet is well submerged, with its upper end 5.5 feet below the water surface. The inlet is 3.5 feet high and 6 feet wide, protected by a vertical screen with $\frac{3}{8}$-inch bars spaced 4 inches center to center. The over-all efficiency was tested using the formula $Q = kA\sqrt{2gH}$ and gave a coefficient of discharge of 0.65. These siphons are not provided with water seals on their outlet ends, but have free discharge. The whole structure, gate included, was placed in a length of 90 feet and the cost was one-third that estimated for the overflow type of spillway.

UNITED STATES RECLAMATION SERVICE SIPHONS.

The United States Reclamation Service has located a siphon about 12 miles below the heading of the Yuma Project, where the main canal leaves the foot of the mesa and turns southward toward Yuma. At this point there is a drop of about 12 feet in water level, bringing the canal to the level of the lower valley. It is intended to develop power here to be delivered to a point below Yuma and used for pumping water from the main canal through a lift of about 80 feet to the top of the mesa, for the irrigation of some 30,000 to 40,000 acres of land. A battery of 5 siphons has been installed. They can be adjusted to discharge at different levels and all employed when the canal is running full, the combined theoretical discharge being 1,488 second-feet. They have been tested operating as a battery at efficiencies ranging from 68 to 70 per cent and in combinations from 64 to 80 per cent. The area of the smallest or throat section, which was used in the computations, is 11.35 square feet, and at the outlet end 21 square feet. In the tests for efficiency the actual drop between water surfaces was 11.87 feet, which was certainly all the head available for producing velocity, but the partial vacuum registered by the mercury gage showed an equivalent of 15.60 feet of water.

This was noted as indicating that the siphon was acting in a manner similar to a compound diverging tube under pressure and having a discharge coefficient greater than one and which may even have been greater than two. It also indicated that the draft tube should flare, which was the case in this instance. The observed depth over the lip of throat necessary to start siphonic action was 0.35, 0.40, 0.35, 0.15, 0.35, and 0.40 foot respectively for the siphons as listed above and with 5.35 feet of water over the lip of the outlet, and they ran uniformly one-tenth foot higher in each case with the outlet sealed with 6 feet of water. They are of reinforced concrete and cost about $23,000 complete. The regulation of siphonic action is by means of a specially designed sliding air valve shown in figure 9. This same design is used on the installations at the Salt River project near Phoenix, Ariz. The operation of these control valves was satisfactory, except that there was not enough vertical movement to permit

of a wide range of regulation of the water surface and floating trash had a tendency to enter and clog the neck of the valves. Screens fixed on their lower ends removed the difficulty.

On the Sun River project there is a structure on the main canal, shown in figures 1 and 2, Plate XIV, and in cross-section in figure 10, which combines a storm culvert, sluice gate, and siphon spillway. The canal at the point of installation has a maximum capacity of 1,000

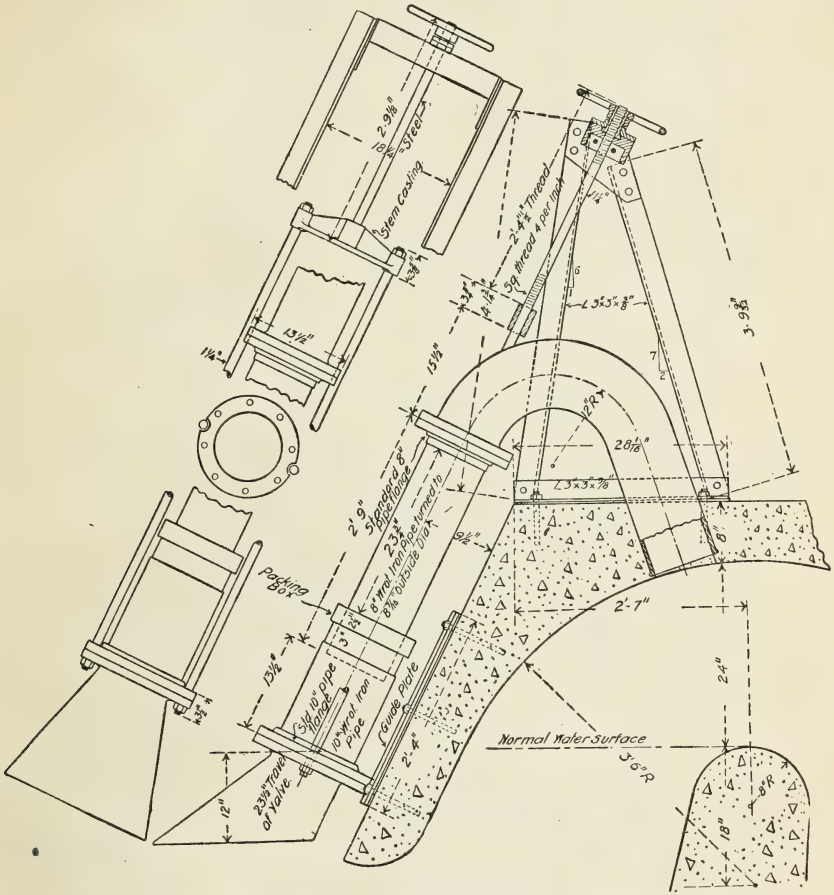


FIG. 9.—Air-control valves to start and stop siphonic action. Installed on siphon spillways of U. S. Reclamation Service near Phoenix and Yuma, Ariz.

second-feet, and the siphon is designed to dispose of any water in excess of that amount up to 1,500 second-feet, because the combination of the three siphons is calculated to discharge 500 second-feet. To provide for any possibility of silt deposit at this point, the three sluice gates, each 3 by 3 feet, operating under a head of 11 feet, will discharge about 500 second-feet and the culvert which conducts the flow to a natural drainage channel is designed for capacities of 900

and 1,400 second-feet at the intake and outlet ends respectively. The siphons are to operate under a head of 12 feet, at an average velocity of about 14 feet per second, assuming 0.50 as a coefficient of discharge.

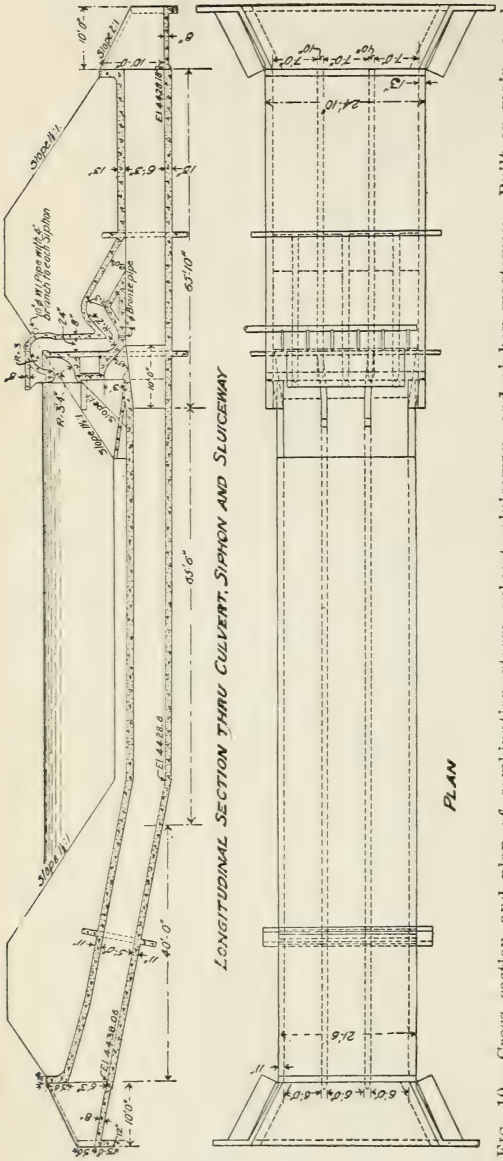


FIG. 10.—Cross section and plan of combination storm culvert, sluiceway, and siphon wasteway. Built on main canal of the U. S. Reclamation Service, Sun River project, Mont.

The structure is of reinforced concrete throughout, with a detached air chamber for the regulation of the siphons. The air control has not given satisfactory results because of its isolation; otherwise the structure is a good example of the combination of facilities for the control of superdrainage on side-hill canal or where melting snows or obstruction below the structure might cause an abnormal rise of the water level in the canal at the site of the siphon.

A structure to regulate the Arizona Canal of the Salt River project, at the Arizona Power Plant, is shown in figure 8. The area of the throat section is 10 square feet, flaring to a maximum section of 15 square feet at a point 10 feet above the lip of the outlet, where it tapers to the same form and cross section as that of the throat. This is the only case of the tapering draft tube known to the writer and it is to be regretted that there is no information as to the efficiency of the structure. Such a series of tests as would give results enabling the engineering profession to ascertain

the effects of this departure and verify or correct the theory of the parting water column in siphons under relatively high heads would be of benefit, and the tests taken on the Yuma installation, where the design is identical except for the converging outlet, could be contrasted.

As a method of automatically safeguarding the freeboard of a canal at isolated points, the small siphon shown in figure 2, Plate XI, and in text-figure 11 is a good example. These figures illustrate the siphon at the head of the East Park Feed Canal of the Orland project. It is designed to operate when the water stands 0.2 foot above the top of the waste weir at the place of diversion, and thus furnish a close regulation of the water surface. The estimated capacity is 99 second-feet, with 0.50 taken as the discharge coefficient. This was one of the first installations in the United States and followed the European custom of inclined draft tube, the slope of the ground at the site being particularly adapted to the design.

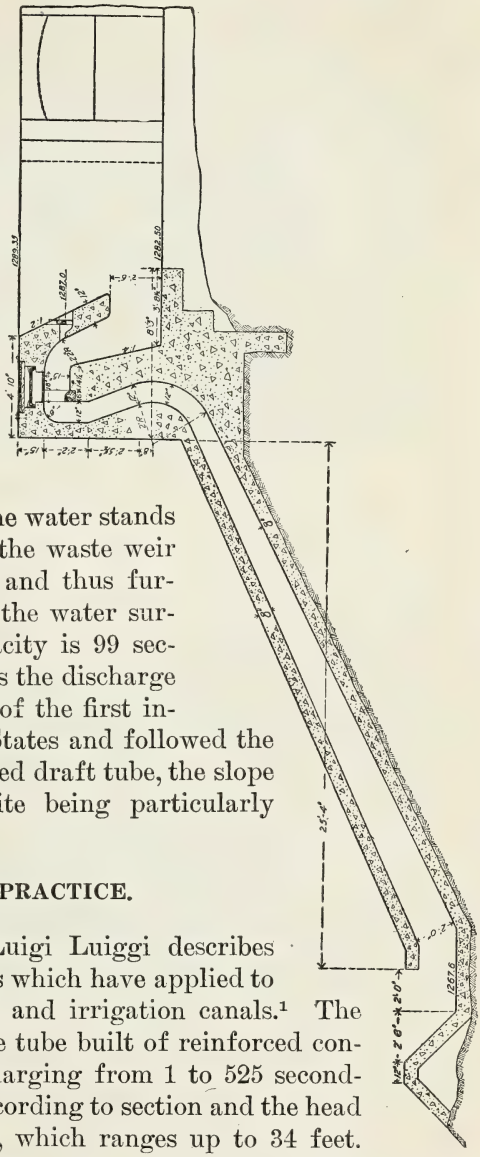


FIG. 11.—Cross section of siphon spillway, headworks East Park supply canal, Orland project, California.

EUROPEAN PRACTICE.

The Italian engineer Luigi Luiggi describes numerous siphon spillways which have applied to dams and to many power and irrigation canals.[1] The prevailing type is a square tube built of reinforced concrete and capable of discharging from 1 to 525 second-feet, varying, of course, according to section and the head under which they operate, which ranges up to 34 feet. To produce larger discharges where head was limited, siphons are placed in batteries. A typical example of clever control by the siphon spillway is shown in the case of the Logalunga Reservoir, near Genoa,

[1] Transactions of the International Engineering Congress, 1915, Waterways and Irrigation Section.

where the old overflow spillway required a head of from 4 to 8 feet to discharge the freshets, and, as is usually the case where the overfall spillway is used, quantities of valuable water ran to waste over the spillway after each storm. A battery of 10 siphons, each with an internal cross section (fig. 6) 6 feet 4 inches square and a working head of 18 feet 11 inches has a calculated capacity of 5,250 second-feet. At the same time the sill of the spillway was elevated 3 feet, producing additional storage for about 240 acre-feet. This additional capacity is a very important advantage in places where water is valuable. It was, moreover, proved that the siphon gave a discharge of 525 second-feet when the water rose to 4 inches over the lip, which was a rise of 14 inches less than with the old spillway, thus reducing the water pressure against the dam correspondingly. The additional storage referred to above represented an income of at least $3,000 per year, and if capitalized at 5 per cent would produce a total of $60,000, or at least six times the cost of the new spillways. There are numerous examples of such possibilities in this country known to the writer where the siphon spillway should be considered.

In one of the large hydroelectric developments in the western part of the United States there was constructed a dam over 100 feet in height with a spillway section close to 600 feet in length. The maximum head on the crest with a discharge of 100,000 second-feet is 14 feet, with a freeboard of 4 feet. In other words, it was necessary to design the structure so as to limit the range of rise in the reservoir to 14 feet above the crest of the spillway and at the same time take care of the maximum inflow. With a battery of spillways of the siphon type such as was installed in the Sweetwater dam or the Huntington Lake dam, in California, at least one-half of the spillway area could have been saved with an additional storage of about 8 feet of water in the top of the reservoir, where each foot means an enormous amount of stored energy. In addition, much closer regulation of the pond level would have been provided.

In Italy the structure has been more fully developed and used, and it is stated that there are more than 100 siphon spillways in use in connection with Italian dams and canals.

HUNTINGTON LAKE SIPHON SPILLWAY.

Huntington Lake is located about 50 miles northeast of Fresno, Calif., and is formed by the construction of three dams impounding the waters of Big Creek, a tributary of the San Joaquin River. It has a watershed area of 80 square miles and an average annual precipitation of about 31 inches. The dams are of concrete, of the gravity arch type, two of them 100 feet in height and one over 150 feet

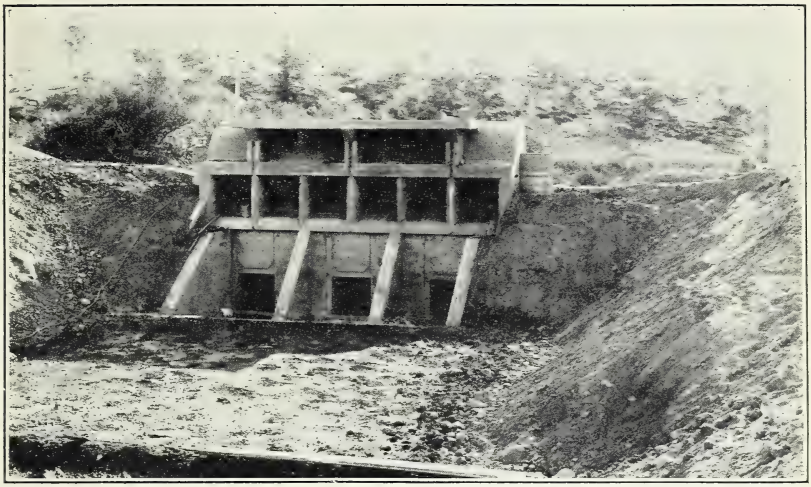


FIG. 1.—INTAKE END OF SIPHON WASTEWAY ON MAIN CANAL OF SUN RIVER PROJECT.

Note the three sluice gate openings referred to in the description.

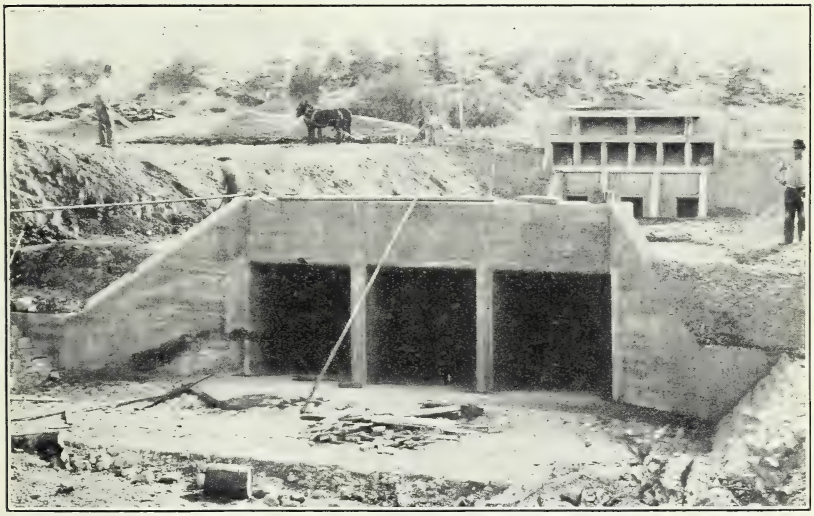


FIG. 2.—INTAKE END OF STORM CULVERT SHOWN IN FOREGROUND AND SIPHONS IN BACKGROUND.

FIG. 1.—OUTLET OPENING FOR OVERPOUR SPILLWAY AT SOUTH END OF SWEETWATER DAM, CALIFORNIA.

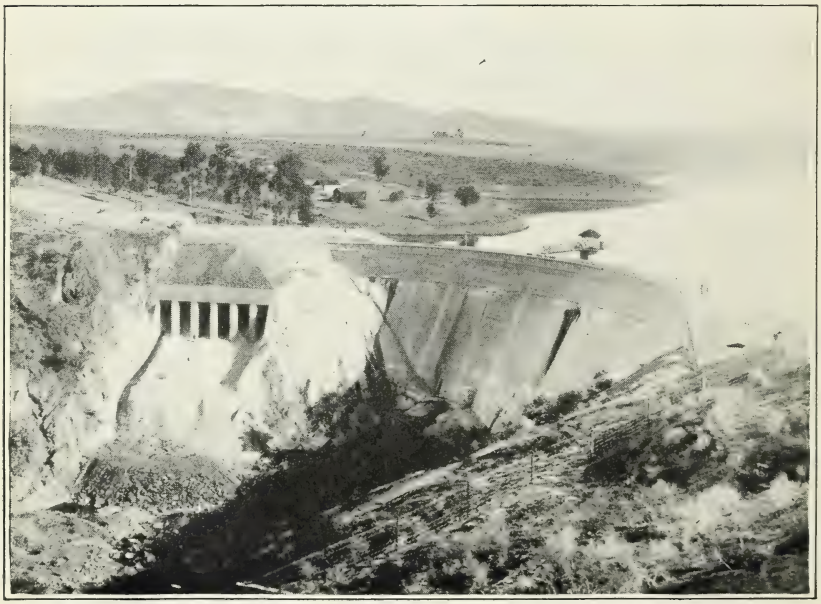


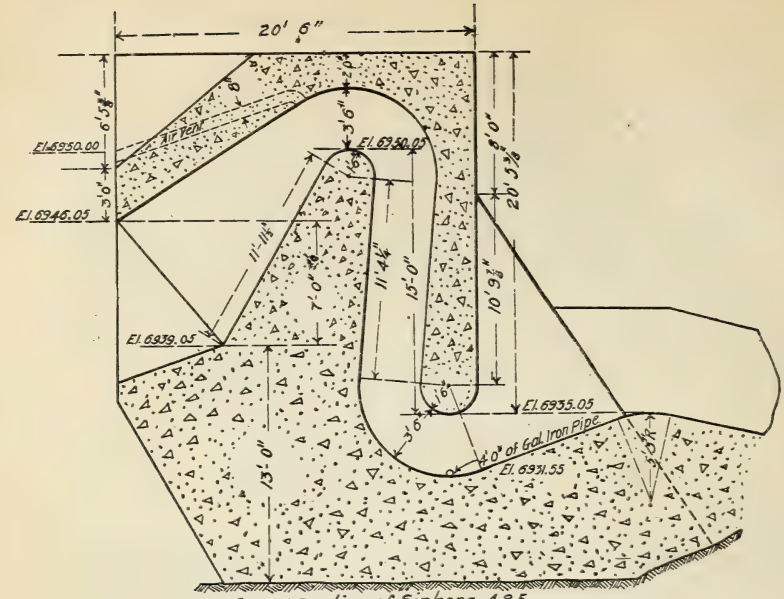
FIG. 2.—SWEETWATER DAM AS COMPLETED AFTER THE FLOOD OF 1915.

The outlet side of the battery of six siphons is shown in the picture on the left-hand side of the dam. This was added as additional spillway after the flood had cut a channel where the siphons are now located.

high, the latter being in Big Creek itself. In 1917 these dams were all raised 35 feet to an elevation of 6,955.5 feet. The dam known as No. 1 was the only one provided with a spillway, the other two being constructed above any possibility of overflow. An overflow crest 645 feet long was built in dam No. 1 over which flow begins after the water surface in the reservoir exceeds an elevation of 6,950 feet. The downstream face was built in a series of steps, each 4 feet high and 2 feet wide, and to prevent any great amount of water going over the dam, a battery of 7 siphons was installed to operate when the water on the spillway crest had reached a depth of 6 inches or more.

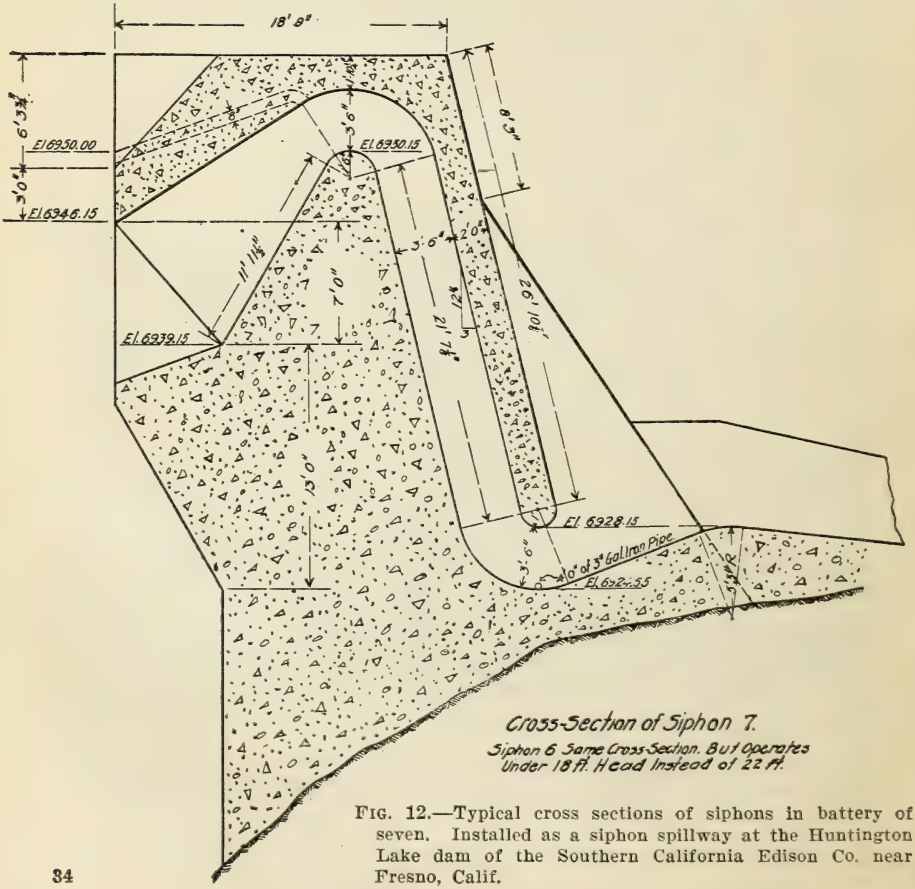
The cross-sections of these siphons are shown in figure 12, and it will be noted that they are not of uniform design in that five of them have the outlet legs inclined toward the intake and two of them slope away from the intake. They are all of reinforced concrete with throats 3.5 feet high and 12 feet wide, containing an area of 42 square feet. Because of the sloping topography of the canyon side where the siphons are built, they also vary as to operating head, three of them being designed to utilize 12 feet, two of them 15 feet, one 18 feet, and one 22 feet fall from head to tail water. The intakes extend 4 feet below normal water surface and are tapered from an area 9.25 feet high by 15.5 feet wide to that at the throat, 42 square feet, and the draft leg is uniformly of the same area as that at the throat. Except for extreme care in making the concrete dense enough to exclude air leaks in the siphons, no other precautions were taken to effect a special lining other than a coat of "gunite," which was applied after the forms were removed. Air control was provided by an 8 by 27 inch air duct connecting the upstream face of the dam with the crown of each siphon chamber, the top of the air inlet being at the same elevation as the overflow spillway crest. A sliding gate is fixed over these air ducts, the vertical movement of which is 16 inches, which makes possible the regulation of the siphons to a point that far below the point where siphonic action would ordinarily cease.

A peculiar condition was developed as a result of the varying heads under which the siphons were to operate in that the sealing basin of the upper, or lower-head, units would tend to overflow into the adjacent basins, causing a rise of water in the siphon chambers at the outlet end and transmitting adverse air pressure to the intake arm and resulting in the lowering of the water there and retarded priming of that particular siphon. To eliminate this condition there was installed a system of relief pipes connected with the crowns of the siphons so that any air pressure in one siphon produced by the action of another could be expelled by the siphon in action. This was sufficient for those whose outlets were of the same level, but the pipes for



Cross-Section of Siphons 4&5.

Siphons 1, 2&3 of Same Cross-Section, but Operate Under 12 ft. Head Instead of 15 ft.



Cross-Section of Siphon 7.

Siphon 6 Same Cross-Section, But Operates Under 18 ft. Head Instead of 22 ft.

FIG. 12.—Typical cross sections of siphons in battery of seven. Installed as a siphon spillway at the Huntington Lake dam of the Southern California Edison Co. near Fresno, Calif.

the others were led to a point 2 inches under water where air pressure could be relieved but back suction after priming was retarded by the high lift. The calculated capacity of battery is 5,200 second-feet and since the project has been but recently completed no tests of efficiency have been conducted. It is evident that the assumed discharge coefficient was conservative. The siphons replace a system of flashboards 6 feet high which were used before the dams were raised.

Figure 13 represents a smaller type placed at a number of points along the canal system of the Mount Whitney Power and Light Company in California. The operating head is 3.5 feet, the cross section 0.5 by 2 feet, and the approximate discharge 10 second-feet. Larger structures installed in batteries of three units are also placed along their canal system (fig. 14), designed along the same general lines, but capable of discharging up to 100 second-feet. The only trouble found with any of these was due to ice and floating trash which clogs the intakes, but which is seldom a great menace, since the structures are rarely in operation during the period of freezing weather. Figure 7 shows the outline of a typical European design such as was installed by Gregotti and as designed to overcome the objection where trouble may be expected from ice and floating trash. The air regulation is so arranged that it can be manipulated to break siphonic action when the ordinary air duct intake is menaced by ice. Provision is also made for the drainage of the water-seal basin to prevent freezing and consequent stopping of the outlet.

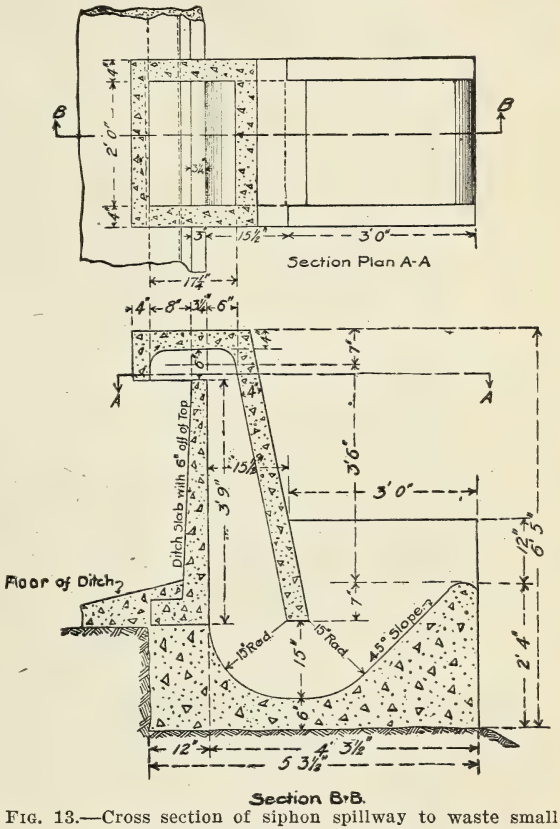


FIG. 13.—Cross section of siphon spillway to waste small amounts of water. Located at isolated points on the canal system of the Mount Whitney Power & Electtic Co. near Visalia, Calif.

The claim that the siphon spillways constructed as control devices on the New York State Barge Canal were the first installations in this country is undisputed. This project offered several ideal cases for testing their adaptability in regulatory capacities and they were utilized in several locations where other and better known structures were either impracticable or impossible. Two of the three locations

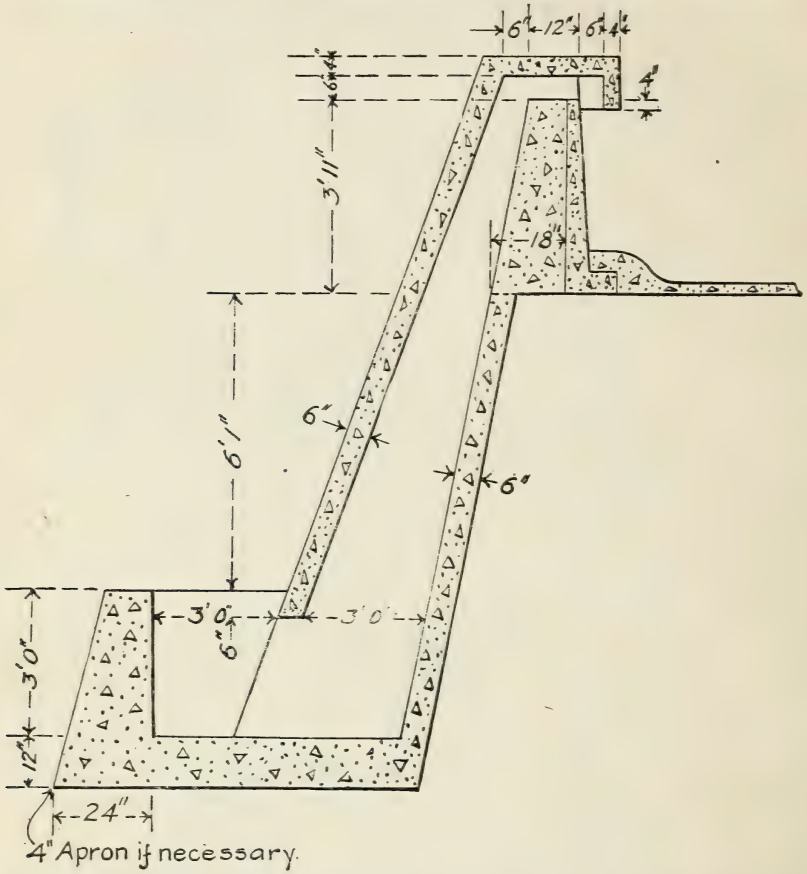


FIG. 14.—Cross section of larger size siphon spillway in battery of three. These are also located at points where automatic operation is essential, and as built and operated by the Mount Whitney Power & Electric Co. are capable of discharging 100 cubic feet per second.

required a structure capable of disposing of the waters accumulated from an intercepted drainage area and which had to be discharged into an adjacent stream. Surface fluctuation was limited, so that the overflow into the canal could not be in excess of the outflow and it had to be discharged with rapidity equal to the inflow. The old form of overpour spillway with the necessary length of crest to dispose of the water within the limits of surface rise and fall was impossible in at

least one case because of the lack of area in which to build it. At this point it was estimated that the maximum inflow from the drainage area was about 700 second-feet, which the siphons must take care of during heavy rainfall and in addition limit the fluctuation of water surface to 1 foot. A weir to discharge the inflow stated, with the limiting head on its crest, required a length of 200 feet which was not permissible under the conditions and a spillway consisting of four siphons and a 20-foot waste weir for floating trash, required but 57 feet between abutments. Each siphon has a throat area 7.75 feet wide by 1 foot high and acts under a head of 10.5 feet calculated to discharge 160 feet. The 20-foot weir with the 1 foot head will discharge 70 second-feet, making a total of 710 second-feet. The inlet of the siphons is well submerged to prevent the entrance of floating bodies and is further protected by screens. The inlets were flared to twice the normal area to reduce to a minimum the loss of head due to entry, and the throat of the siphons were each made 1 foot high, because it was desired to retard full siphonic action until the tubes were completely filled at that point. They were flared again to a section of 2 feet by 4 feet at the outlet end. Air vents 6 inches high by 12 inches long were provided for each siphon, piercing the wall at low-water level, thus regulating the action at that point.[1]

Contrasting examples of requirements of overflow and siphon spillways are furnished in Italy, where the limiting lengths of spillway sites are in a ratio of about 6 to 1. The two plants in question are a hydraulic power plant near Milan, which is equipped with ordinary overflow spillway, and another at Verona with a siphon spillway. The siphon is confined within a space 59 feet in length and limits the surface fluctuations to 3 inches, while the overflow spillway at Turbigo, near Milan, is 300 feet in length and does not discharge the required volume until 2 feet of water pours over its crest. In addition the overflow spillway is supplemented by three gates automatically worked by electric motors.[2] The conclusion as to control devices is self-evident.

The battery of 6 siphons recently completed for the spillway at the north end of Sweetwater Dam, California (fig. 2, Pl. XV), is additional to the old overflow spillway on the south end, which in itself has been remodeled and extended as a result of the damage by the flood of 1915. There is also an emergency overflow spillway 500 feet long in the center of the dam. The siphon spillway is the largest of the type constructed to date, having an intake area for each unit of 144 square feet, a throat area 6 by 12 feet, or of 72 square feet, an outlet opening 8.5 by 12 feet, or of 102 square feet, and operating

[1] Engineering News, vol. 64, No. 15. [2] Engineering News, Apr. 20, 1911.

under a minimum head of 36 feet. The calculated discharge of the battery is 16,600 second-feet with an assumed coefficient of 0.70 and a minimum head of 36 feet. The total capacity of the siphons and the overflow spillways at the south end and center is now estimated to be 50,000 second-feet with 5 feet of water flowing over the center spillway. Figure 1, Plate XV, shows a cut of the south-end spillway. About 300 feet below the dam proper there is a reinforced concrete hollow-type dam built to provide a stilling pool and neutralize the energy of the water falling from the three spillways.

CONCLUSIONS.

The purpose of this bulletin is to assemble, as fully as possible, all the best information known to the writer in such a way as to be of use to anyone interested in the subject. It is the intention of the writer to point out, in as brief a manner as he can, the advantages of the siphon spillway when it is desired to facilitate the escape of high flood crests, and at the same time to conserve crest length, and cost of construction and maintenance, by eliminating the use of mechanical or other energy necessary to operate partially or completely automatic spillways of other types.

It is not too broad a statement to say that the siphon is the only absolutely "foolproof" method of maintaining adequate spillway capacity without the addition of moving parts to be constantly cared for and frequently replaced, and which very often fail to operate automatically or because of the absence of an intended operator.

Furthermore, the siphon permits of (1) closer regulation of the pond level, (2) the heightening of the spillway crest and therefore additional storage capacity where each unit of height is of greater value, (3) the coming into full and efficient action almost at the moment the danger point is reached, instead of having to depend upon additional danger-producing head to increase discharge, and (4) maintaining the desired pond level by quickly ceasing to act when the danger is passed.

It is evident that having provided adequate spillway of the siphonic type in about one-half the area required for any other known automatic type, the escaping water is concentrated to a narrow jet. This allows a more economical arrangement of the channel intended to neutralize the accelerated flow and convey it to a natural drainage course.

The writer has never heard any question raised as to the possibility of damage to land or structures below the point of installation of a siphon spillway, but it has occurred to him that such possibilities do exist and that provisions may be considered to counteract such damage. The spillway of a reservoir may be called upon to discharge a large volume of water.

In cases where ordinary overflow spillways are operating, the water released and allowed to continue down the watercourse would increase and decrease with the head on the overflow, and would at no time during the discharge be greater than the volume of water contributed to the reservoir above the structure. The watercourse under such conditions would be performing its natural functions.

In the case of an ordinary siphon spillway, from the instant the siphon comes into action the volume in the channel below the structure becomes the full spillway capacity and the burden of conveying this suddenly imposed surplus may be compared to that of a similarly released flood; this for the reason that the spillway capacity is usually assumed to care for both normal and flood flow into the reservoir.

A battery of siphons may be regulated so as to bring each unit into action separately or in pairs by placing their crests and air intakes at different elevations. The regulating parts are usually placed near the high-water line of the reservoir, where flow into it results in a slow-rising water surface.

Varying the elevation of the priming parts of the different units need not utilize a range of more than from 1 to 2 feet in height, while it would regulate the outflow to conform more closely to the conditions of an ordinary overflow spillway and the volume released to the watercourse below the dam would vary with the reservoir inflow.

The lower unit should be fixed to operate at the level where it is desired to maintain the reservoir surface, and the remaining units should have their crests and air intakes set to operate at slightly higher elevations—still maintaining a safe freeboard above the highest air vent.

The writer is of the opinion that this practice has been followed on several of the larger installations, but for the purpose of regulating the pond level and not to govern the discharge to the stream below.

There are conditions where the siphon spillway is not adapted to the site or to the requirements which it is intended to serve, and its failure to perform under the improper conditions has led to condemnation of the structure as a type. For instance, one case cited in a criticism to the writer, refers to the tendency of a siphon to check the velocity of the stream near its intake and encourage the deposit of silt in front of the structure, where it was used as a regulator at the end of a canal. The design of this siphon did not take into consideration its utilization as a scouring device, but was intended only to skim the surface water and prevent the overflow of the banks of the canal. Some provisions should have been made to temporarily take care of the silt problem, since it was intended, as an ultimate development, that a power plant would be located near the site of

the siphon and its installation was never intended for other than regulatory purposes. It may be stated that any other nonautomatic method of control would have presented the same objections found in this case.

Another case where the siphon was condemned as a failure proved to be its attempted use as a series of " drops " on a canal system, and the principal objection in this instance was its failure to deposit the water from the upper to the lower channel in a vertical rather than a horizontal direction, necessitating more protection to prevent scour below the structure.

It is agreed that the use of a siphon as a substitute for the ordinary overpour drop would conserve area and in addition would deliver vastly more water to the lower level per unit of cross section, but it would also produce a velocity at the outlet so much greater than the velocity developed over an overpour drop that some method of destroying the energy of the rapidly falling water must be provided. It is not evident, however, why the water should not emerge from the siphon in vertical direction as it does in an ordinary drop. It is the opinion of the writer that the use of the siphon as a drop is another case of applying the principle to a foreign use without taking the necessary precautions to provide for its primary factor—accelerated velocity.

It is hoped that in the near future the more complete study of the structure as a standard design for different conditions, forms, and efficiencies will develop data to permit of its extended use.

ACKNOWLEDGMENTS.

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